Materials and Structure

1878

CHAPTER IV.

OF CARPENTRY.

Section I.—Of Timber.

299. **Structure of Timber.**—Timber is the material of trees belonging almost exclusively to that class of the vegetable kingdom in which the stem grows by the formation of successive layers of wood all over its external surface, and is therefore said by botanists to be *exogenous*.

The exceptions are, trees of the palm family, and tree-like grasses, such as the bamboo, which belong to the *endogenous* class; so called because, although the stem grows partly by the formation of layers of new wood on its outer surface, the fibres of that new wood do nevertheless cross and penetrate amongst those previously formed in such a manner as to be mixed with them in one part of their course, and internal to them at another.

The stems of endogenous trees, though light and tough, are too flexible and slender to furnish materials suitable for important works of carpentry. They will therefore not be further mentioned in this section except to refer to the tables at the end of the volume for the tenacity and heaviness of bamboo.

The stem of an exogenous tree is covered with bark, which grows by the formation of successive layers on its inner surface, at the same time that the wood grows by the formation of successive layers on its outer surface. This double operation takes place in the narrow space between the previously-formed wood and bark, during the circulation of the sap. The sap ascends from the roots to the leaves through vessels contained in the outer layers of the wood; at the surface of the leaves it acquires carbon from the atmosphere, and becomes denser, thicker, and more complex in its composition; it then descends from the leaves to the roots through vessels contained chiefly in the innermost layers of the bark. It is believed that the formation of new wood and bark takes place either wholly or principally from the descending sap.

The circulation of the sap is either wholly or partially suspended during a portion of each year (in tropical climates during the dry season, and in temperate and polar climates during the winter); and hence the wood and bark are usually formed in distinct layers,

at the rate of one layer in each year; but this rule is not universal. Each such layer consists of parts differing in density and colour to an extent which varies in different kinds of trees.

The tissues of which both wood and bark consist are distinguished into two kinds—*cellular tissue*, consisting of clusters of minute cells; and *vascular tissue*, or *woody fibre*, consisting of bundles [illegible] nder tubes; the latter being distinguished from the former [illegible] its fibrous appearance. The difference, however, between those two kinds of tissue, although very distinct both to the eye and to the touch, is really one of degree rather than of kind; for the fibres or tubes of vascular tissue are simply very much elongated cells, tapering to points at the ends, and "breaking joint" with each other.

The tenacity of wood when strained "along the grain" depends on the tenacity of the walls of those tubes or fibres; the tenacity of wood when strained "across the grain" depends on the adhesion of the sides of the tubes and cells to each other. Examples of the difference of strength in those different directions will be given afterwards.

When a woody stem is cut across, the cellular and vascular tissue are seen to be arranged in the following manner:—

In the centre of the stem is the *pith*, composed of cellular tissue, enclosed in the medullary sheath, which consists of vascular tissue of a particular kind. From the pith there extend, radiating outwards to the bark, thin partitions of cellular tissue, called *medullary rays;* between these, additional medullary rays extend inwards from the bark to a greater or less distance, but without penetrating to the pith.

When the medullary rays are large and distinct, as in oak, they are called "*silver grain.*"

Between the medullary rays lie bundles of vascular tissue, forming the woody fibre, arranged in nearly concentric rings or layers round the pith. These rings are traversed radially by the medullary rays. The boundary between two successive rings is marked more or less distinctly by a greater degree of porosity, and by a difference of hardness and colour.

The annual rings are usually thicker at that side of the tree which has had most air and sunshine, so that the pith is not exactly in the centre.

The wood of the entire stem may be distinguished into two parts—the outer and younger portion, called "*sap-wood,*" being softer, weaker, and less compact, and sometimes lighter in colour, than the inner and older portion, called "*heart-wood.*" The heart-wood is alone to be employed in those works of carpentry in which strength and durability are required. The boundary

between the sap-wood and the heart-wood is in general distinctly marked, as if the change from the former to the latter occurred in the course of a single year. The following examples of the proportion of sap-wood to the entire volume are given on the authority of Tredgold. (*Principles of Carpentry*, Section X.)

Tree.	Age. Years.	Diameter. Inches	Rings of Sap-wood.	Thickness of Sap-wood. Inches.	Proportion of Sap-wood to whole Trunk
Chestnut, ...	58	$15\frac{1}{2}$	7	$\frac{3}{8}$	0·1
Oak, ...	65	17	17	$1\frac{1}{4}$	0·294
Scotch Fir, ...	?	24	?	$2\frac{1}{2}$	0·416

The following data are given on the authority of Mr. Robert Murray, C.E. (*Encyc. Brit.*, Article "Timber.")

Tree.	Rings of Sap-wood.
English Oak (*Quercus pedunculata*), ...	12 to 15
Durmast Oak (*Quercus sessiliflora*), ...	20 to 30
Chestnut (*Castanea Vesca*), ...	5 or 6
Elm (*Ulmus campestris*), ...	about 10
Larch (*Larix Europæa*), ...	" 15
Scotch Fir (*Pinus sylvestris*), ...	" 30
Memel Fir (*Pinus sylvestris*), ...	" 44
Canadian Yellow Pine (*Pinus variabilis*), ...	" 42

The structure of a *branch* is similar to that of the trunk from which it springs, except as regards the difference in the number of annual rings, corresponding to the difference of age. A branch becomes partially imbedded in those layers of the trunk which are formed after the time of its first sprouting; it causes a perforation in those layers, accompanied by distortion of their fibres, and constitutes what is called a *knot*. (On various matters mentioned in this Article, see Balfour's *Manual of Botany*, Part I., chaps. i. and ii.)

300. **Timber Trees Classed—Pine-wood - Leaf-wood.**—For purposes of carpentry trees may be classed according to the mechanical structure of the wood. It has already been stated that the botanical classes of Endogens and Exogens correspond to essential differences of mechanical structure.

In further dividing the class of Exogenous trees, or timber-trees proper, according to the structure of the wood, a division into two classes at once suggests itself, which exactly corresponds with a botanical division, viz. :—

Pine-wood, comprising all timber-trees belonging to the coniferous order; and

Leaf-wood, comprising all other timber-trees.

Beyond this primary division, the place of a tree in the botanical system has little or no connection with the structure of its timber.

A classification of timber according to its mechanical structure was proposed by Tredgold, founded, in the first place, on the greater or less distinctness of the medullary rays; and secondly, on the greater or less distinctness of the annual rings. According to that classification, pine-wood, or coniferous timber, is placed in the same class with leaf-wood that has the medullary rays indistinct; and this is certainly a fault in the system. If, however, pine-wood be placed in a class apart, Tredgold's system may very well be applied to divide and subdivide the class of leaf-wood; but it is to be observed that the characters on which that system is founded, being mere differences in degree, and not in kind, are not of that definite sort which a thoroughly satisfactory system of classification requires; and if they are adopted, it is because no better set of distinguishing characters has yet been proposed.

The following is a condensed view of the classification of exogenous timber, as above described:—

CLASS I. PINE-WOOD. (Natural order *Coniferæ*.)—Examples; —Pine, Fir, Larch, Cowrie, Yew, Cedar, Juniper, Cypress, &c.

CLASS II.—LEAF WOOD. (Non-coniferous trees.)

DIVISION I. With distinct large medullary rays. (The trees in this division form part of the natural order *Amentaceæ*.)

Subdivision I. Annual rings distinct.—Example:—Oak.

Subdivision II. Annual rings indistinct.—Examples: —Beech, Alder, Plane, Sycamore, &c.

DIVISION II. No distinct large medullary rays.

Subdivision I. Annual rings distinct.—Examples: —Chestnut, Ash, Elm, &c.

Subdivision II. Annual rings indistinct.—Examples: —Mahogany, Walnut, Teak, Poplar, Box, &c.

The chief practical bearings of this classification are as follows:—

Pine-wood, or coniferous timber, in most cases, contains turpentine. It is distinguished by straightness in the fibre and regularity in

the figure of the trees; qualities favourable to its use in carpentry, especially where long pieces are required to bear either a direct pull, or a transverse load, or for purposes of planking. At the same time, the lateral adhesion of the fibres is small; so that it is much more easily shorn and split along the grain, or torn asunder across the grain, than leaf-wood; and is therefore less fitted to resist thrust or shearing stress, or any kind of stress that does not act along the fibres. Even the toughest kinds of pine-wood are easily wrought. A peculiar characteristic of pine-wood (but one which requires the microscope to make it visible) is that of having the vascular tissue "*punctated;*" that is to say, there are small lenticular hollows in the sides of the tubular fibres. This structure is probably connected with the smallness of the lateral adhesion of those fibres to each other.

In Leaf-wood, or non-coniferous timber, there is no turpentine. The degree of distinctness with which the structure is seen, whether as regards medullary rays or annual rings, depends on the degree of difference of texture of different parts of the wood. Such difference tends to produce unequal shrinking in drying; and consequently those kinds of timber in which the medullary rays, and the annual rings, are distinctly marked, are more liable to warp than those in which the texture is more uniform. At the same time, the former kinds of timber are, on the whole, the more flexible, and in many cases are very tough and strong, which qualities make them suitable for structures that have to bear shocks.

301. **Appearance of good Timber.**—There are certain appearances which are characteristic of strong and durable timber, to what class soever it belongs.

In the same species of timber, that specimen will in general be the strongest and the most durable which has grown the slowest, as shown by the narrowness of the annual rings.

The cellular tissue as seen in the medullary rays (when visible) should be hard and compact.

The vascular or fibrous tissue should adhere firmly together, and should show no wooliness at a freshly-cut surface, nor should it clog the teeth of the saw with loose fibres.

If the wood is coloured, darkness of colour is in general a sign of strength and durability.

The freshly-cut surface of the wood should be firm and shining, and should have somewhat of a translucent appearance. A dull, chalky appearance is a sign of bad timber.

In wood of a given species, the heavier specimens are in general the stronger and the more lasting.

Amongst resinous woods, those which have least resin in their

pores, and amongst non-resinous woods, those which have least sap or gum in them are in general the strongest and most lasting.

It is stated by some authors that in pine-wood, that which has most sap-wood, and in leaf-wood, that which has least, is the most durable; but the universality of this law is doubtful.

Timber should be free from such blemishes as clefts, or cracks radiating from the centre; "cup-shakes," or cracks which partially separate one annual layer from another; "upsets," where the fibres have been crippled by compression; "rind-galls," or wounds in a layer of the wood, which have been covered and concealed by the growth of subsequent layers over them; and hollows or spongy places, in the centre or elsewhere, indicating the commencement of decay.

302. **Examples of Pine-wood,' Pine, Fir, Larch, Cowrie, Cedar, &c.** —The following are examples of timber of this class:—

I. PINE timber of the best sort is the produce of the Red Pine, or Scottish Fir (*Pinus sylvestris*), grown in Norway, Sweden, Russia, and Poland. The best is exported from Riga, the next from Memel and from Dantzic. The same species of tree grows also in Britain, but is inferior in strength. The annual rings, when this timber is of the best kind, consist of a hard part, of a clear dark-red colour, and a less hard part, of a lighter colour, but still clear and compact. The thickness of the rings should not exceed one-tenth of an inch. The most common size of the logs to be met with in the market is about 13 inches square. This is the best of all timber for straight beams, straight ties, and straight pieces in framework generally, and for the spars of ships.

Pine timber for the same purposes is also obtained from various other species, chiefly North American, of which the best are the Yellow Pine (*Pinus variabilis*), and White Pine (*Pinus Strobus*). It is softer and less durable than the Red Pine of the North of Europe, but lighter, and can be had in larger logs.

II. WHITE FIR, or DEAL timber of the best kind, is the produce of the Spruce Fir (*Abies excelsa*), grown in Norway, Sweden, and Russia. The best is that known as Christiania Deal. Much of this timber is sawn up for sale into pieces of various thicknesses suited for planking, which,

when 7 inches broad are called			"*battens*."
when 9	,,	,,	"*deals*."
when 11	,,	,,	"*planks*."

They are to be had of various lengths; but the most usual length is about 12 feet.

This is an excellent kind of timber for planking, light framing, and joiners' work, and for the lighter spars of ships.

Amongst other kinds of spruce fir, applied to the same purposes, are the North American White Spruce (*Abies alba*), and Black Spruce (*Abies nigra*).

III. The LARCH (*Larix Europæa*), grown in various parts of Europe, furnishes timber of great strength, and remarkable for durability when exposed to the weather; but harder to work and more subject to warp than red pine. The best sort has the harder part of the rings of a dark-red, and the softer part of a honey-yellow; and its rings are somewhat thicker than those of red pine.

Two North American species, the Black Larch, or Hackmatack (*Larix pendula*), and the Red Larch (*Larix microcarpa*), produce timber similar to that of the European Larch.

IV. The COWRIE or KAWRIE (*Dammara Australis*), a coniferous tree, grown in New Zealand, produces timber similar in its properties to the best kinds of pine, except that it is said to be more liable to warp, and more variable in quality. It is of a brownish-yellow colour, and more uniform in its texture than red pine and larch.

V. The term CEDAR is applied, not only to the timber of the true Cedar (*Cedrus Libani*), but also to that of various large species of Juniper (such as *Juniperus Virginiana*) and of Cypress. All these kinds of wood are remarkable for durability, in which they excel all other timber; but they are deficient in strength.

303. **Examples of Leaf-wood—Oak, Beech, Alder, Plane, Sycamore.** —The kinds of timber which head this article belong to the first division of Tredgold's system, being that in which there are distinct large medullary rays. Of the examples cited, the Oak alone belongs to the first subdivision, in which the divisions between the annual rings are distinctly marked by circles of pores. The other examples belong to the second subdivision, in which the rings are less distinctly marked.

I. OAK timber, the strongest, toughest, and most lasting of those grown in temperate climates, is the produce of various species or varieties of the botanical genus *Quercus*. In Europe there are two kinds of oak trees; and it is doubtful whether they are distinct species or varieties of one species. They are:—

The old English Oak, or Stalk-fruited Oak (*Quercus Robur*, or *Quercus pedunculata*), in which the acorns grow on stalks, and the leaves close to the twig, and

The Bay Oak, or Cluster-fruited Oak (*Quercus sessiliflora*), in which the acorns grow in close clusters, and the leaves have short stalks.

Both those kinds of oak come to their greatest perfection in Britain.

The wood of the stalk-fruited oak is lighter in colour, and has more numerous and distinct medullary rays than that of the

cluster-fruited oak, in which they are sometimes so few and indistinct as to have caused it in some old buildings to be mistaken for chestnut. The stalk-fruited oak is the stiffer and the straighter-grained of the two, the easier to work, and the less liable to warp; it is therefore preferable where stiffness and accuracy of form are desired; the cluster-fruited oak is the more flexible, which gives it an advantage where shocks have to be borne.

The best oak timber when new is of a pale brownish-yellow, with a perceptible shade of green in its composition, a firm and glossy surface, very small and regular annual rings, and hard and compact medullary rays. Thick rings, many large pores, a dull surface, and a reddish, or "foxy" hue, are signs of weak and perishable wood.

It is considered that oak timber comes to maturity at the age of 100 years, at which period each tree produces on an average about 75 cubic feet of timber; and that it should not be felled before the sixtieth year of its age, nor later than the 200th.

The species of oak in North America are very numerous. The best of them are, the Red Oak (*Quercus rubra*), and White Oak (*Quercus alba*), which are little inferior to the best European kinds, and the Live Oak (*Quercus virens*) which is said to be superior in strength, toughness, and durability, to all other species, but is so rare as to be reserved exclusively for ship-building.

The wood of the oak contains gallic acid, which probably contributes to the durability of the timber, but tends to corrode iron fastenings.

The following are examples of trees belonging to the second subdivision:—

II. Beech (*Fagus sylvatica*), common in Europe;

III. Alder (*Alnus glutinosa*), also common in Europe;

IV. American Plane (*Platanus occidentalis*), common in North America.

V. Sycamore (*Acer Pseudo-platanus*), also called Great Maple, and in Scotland and the North of England, Plane; common in western Europe.

All these afford compact timber of uniform texture. They are not used for great works of carpentry; but are valuable where blocks of wood are required to resist a crushing force. They last well when constantly wet, and are therefore suited for piles that are to be always under water; but when alternately wet and dry they decay rapidly.

304. **Leaf-wood continued.—Chestnut, Ash, Elm.**—The examples of timber in this article belong to the first subdivision of the second division, according to Tredgold's system, having no large distinct medullary rays, and having the divisions between the

annual rings distinctly marked by a more porous structure. They are in general strong, but flexible.

I. The CHESTNUT (*Castanea vesca*) yields timber resembling that of the cluster-fruited oak, except that it is without large medullary rays, and has less sap-wood. Its properties resemble those of oak timber, except that the chestnut timber is less durable, especially when obtained from old trees.

II. The ASH (*Fraxinus excelsior*) furnishes timber whose toughness and flexibility render it superior to that of all other European trees for making handles of tools, shafts of carriages, and the like; but which is not sufficiently stiff and durable to be used in great works of carpentry. The colour of the wood is like that of oak, but darker, and with more of a greenish hue; the annual rings are broader than those of oak, and the difference between their compact and porous parts more marked.

III. The common ELM (*Ulmus campestris*) and Smooth-leaved Elm (*Ulmus glabra*) yield timber which is valued for its durability when constantly wet, and is specially suited for piles and for planking in foundations under water. Its strength across the grain, and its resistance to crushing, are comparatively great; and these properties render it useful for some parts of mechanism, such as naves of cart wheels, shells of ships' blocks, and the like. It is not suited for great works of carpentry. There are other European species of elm, such as the Wych Elm (*Ulmus montana*), but their timber is inferior to that of the two species named.

A North-American species, the Rock Elm, is said to be not only durable under water, but straight-grained and tough, so as to be well suited for long beams and ties.

305. **Leaf-wood continued.—Mahogany, Teak, Greenheart, Mora.** —These kinds of timber are examples of the second subdivision of Tredgold's second division, having no large distinct medullary rays, and no distinct difference of compactness in the rings. This uniformity of structure is accompanied by comparative freedom from warping.

I. MAHOGANY (*Swietenia mahogani*) is produced in Central America and the West Indian Islands, that of the former region being commonly known as "Bay Mahogany;" that of the latter as "Spanish Mahogany." When of good quality, it is very strong in all directions, very durable, and preserves its shape under varying circumstances as to heat and moisture better than any other kind of timber which can be procured in equal abundance. Mahogany varies much in quality; bay mahogany being in general superior to Spanish mahogany in strength, stiffness, and durability, and in the size of the logs. Spanish mahogany is the more highly valued for ornamental purposes.

Spanish mahogany is distinguished by having a white chalky substance in its pores, those of bay mahogany being empty.

II. Teak (*Tectona grandis*), from its great strength, stiffness, toughness, and durability, is the most valuable of all woods for carpentry, especially for ship-building. It is produced in the mountainous districts of south-eastern Asia and the East India Islands. The best comes from Malabar, Ceylon, Johore, and Java.

Good teak resembles oak in colour and lustre, is very uniform and compact in texture, and has very narrow and regular annual rings. It contains a resinous, oily matter in its pores, in order to extract which, the tree is sometimes tapped; but this injures the strength and durability of the timber, and ought to be avoided. Insects do not attack teak, and iron is not corroded by contact with it, unless it has been grown in a marshy soil.

III. Greenheart (*Nectandra Rodiæi*), a tree of British Guiana, yields a very strong and durable timber, considered of the first quality for ship-building and all kinds of carpentry, and also for piled foundations and other structures under water.

IV. Mora (*Mora excelsa*), also a tree of British Guiana, yields a first-class timber for ship-building.

306. **Leaf-wood continued.—Iron-bark, Blue-gum, Jarrah.**—These are three of the numerous species of the genus *Eucalyptus*, peculiar to Australia. They yield timber of great size, strength, and durability; and that of the iron-bark, in particular, is held to be of the first class for ship building. The wood of iron-bark is white or yellowish; that of blue-gum, straw-coloured; that of jarrah resembles mahogany, and is sometimes called "Australian Mahogany." The *Eucalypti*, in common with some other Australian trees, are distinguished from the trees of other quarters of the globe by being more easily split in concentric layers, than in planes radiating from the pith; and the most frequent blemish in their timber is the occurrence of cylindrical clefts of that kind, filled with gum.

307. **Influence of Soil and Climate on Trees.**—Most timber trees are capable of flourishing in a great variety of soils. The best soil for all of them is one which, without being too dry and porous, allows water to escape freely, such as gravel mixed with sandy loam.

The most injurious soil to trees is that of swampy ground containing stagnant water: it never fails to make the timber weak and perishable.

As to the influence of climate, two general laws seem to prevail: that the strongest timber is yielded, amongst *different species* of trees, by those produced in tropical climates; and amongst trees of *the same species*, by those grown in cold climates. The first law is

exemplified in such woods as teak, iron-wood, ebony, and lignum-vitæ, surpassing in strength all those of temperate climates: the second, in the red pine of Norway, as compared with that of Scotland, in the oak of Britain as compared with that of Italy, and even in the oak of Scotland and the North of England, as compared with that of the South of England.

308. **Age and Season for felling Timber.**—There is a certain age of maturity at which each tree attains its greatest strength and durability. If cut down before that age, the tree, besides being smaller, contains a greater proportion of sap-wood, and even the heart-wood is less strong and lasting. If allowed to grow much beyond that age, the centre of the tree begins either to become brittle, or to soften, and a decay commences by slow degrees, which finally renders the heart hollow. The age of maturity is therefore the best age for felling the tree to produce timber. The following data respecting it are given on the authority of Tredgold:—

	Age of Maturity. Years.
Oak,	60 to 200, average 100
Ash, Elm, Larch,	50 to 100
Fir,	70 to 100

The best season for felling timber is that during which the sap is not circulating—that is to say, the winter, or in tropical climates, the dry season; for the sap tends to decompose, and so to cause decay of the timber. The best authorities recommend also, as a means of hardening the sap-wood, that the bark of trees which are to be felled should be stripped off in the preceding spring.

Immediately after timber has been felled, it should be *squared*, by sawing off four "slabs" from the log, in order to give the air access to the wood and hasten its drying. If the log is large enough, it may be sawn into quarters.

309. **Seasoning, Natural and Artificial.**—Seasoning timber consists in expelling, as far as possible, the moisture which is contained in its pores.

Natural Seasoning is performed simply by exposing the timber freely to the air in a dry place, sheltered, if possible, from sunshine and high winds. The seasoning yard should be paved and well drained, and the timber supported on cast iron bearers, and piled so as to admit of the free circulation of air over all the surfaces of the pieces.

Natural seasoning to fit timber for carpenters' work usually

occupies about two years; for joiners' work, about four years; but much longer periods are sometimes employed.

To steep timber in water for a fortnight after felling it extracts part of the sap, and makes the drying process more rapid.

The best method of *Artificial Seasoning* consists in exposing the timber in a chamber or oven to a current of hot air. In Mr. Davison's process, the current of air is impelled by a fan at the rate of about 100 feet per second; and the fan, air-passages, and chamber are so proportioned, that one-third of the volume of air in the chamber is blown through it per minute. The best temperature for the hot air varies with the kind and dimensions of the timber; thus, for

Oak, of any dimensions, the temperature should not exceed	105° Fahr.
Leaf-woods in general, in logs or large pieces,..	90° to 100°
Pine woods, in thick pieces,................ ..	120°
,, in thin boards,.....................	180° to 200°
Bay mahogany, in boards one inch thick,...	280° to 300°

The time required for drying is stated to be as follows:—

Thickness in inches,........................	1,	2,	3,	4,	6,	8;
Time in weeks,..............................	1,	2,	3,	4,	7,	10,

the current of hot air being kept up for *twelve hours per day* only.

The drying of timber by hot air from a furnace has also been practised successfully by Mr. James Robert Napier, in a brick chamber, through which a current is produced by the draught of a chimney. The equable distribution of the hot air amongst the pieces of timber is insured by introducing the hot air close to the roof of the chamber, and drawing it off through holes in the floor into an underground flue. The hot air on entering, being more rare than that already in the chamber, which is partially cooled, spreads into a thin stratum close under the roof, and gradually descends amongst the pieces of wood to the floor. The air is introduced at the temperature of 240° Fahr. The expenditure of fuel is at the rate of 1 lb. of coke for every 3 lbs. of moisture evaporated.

Many experiments have been made on the loss of weight and shrinkage of dimensions undergone by timber in seasoning; of which the details may be found in the works of Fincham on *Ship-building*, Tredgold on *Carpentry*, Mr. Murray on *Ship-building*,

&c. The results of these experiments vary so much that it is almost impossible to condense them into any general statement. The following shows the limits within which they generally lie:—

Timber.		Loss of Weight per Cent.	Transverse Shrinking per Cent.
Red Pine,	from	12 to 25	$2\frac{1}{2}$ to 3
American Yellow Pine,	,,	18 to 27	2 to 3
Larch,	,,	6 to 25	2 to 3
Oak (British),	,,	[illegible] to 30	about 8
Elm ,,	,,	about 40	about 4
Mahogany,	,,	16 to 25	

310. **Durability and Decay of Timber.**—All kinds of timber are most lasting when kept constantly dry, and at the same time freely ventilated.

Timber kept constantly wet is softened and weakened; but it does not necessarily decay. Various kinds of timber, some of which have been already mentioned, such as elm and beech, possess great durability in this condition.

The situation which is least favourable to the duration of timber is that of alternate wetness and dryness, or of a slight degree of moisture, especially if accompanied by heat and confined air. For pieces of carpentry, therefore, which are to be exposed to these causes of decay, the most durable kinds of timber only are to be employed, and proper precautions are to be taken for their preservation.

Slaked lime hastens the decay of timber, which should therefore, in buildings, be protected against contact with the mortar.

Timber exposed to confined air alone, without the presence of any considerable quantity of moisture, decays by "*dry rot*," which is accompanied by the growth of a fungus, and finally converts the wood into a fine powder.

The following table shows the comparative durability of some kinds of timber for ship-building, as estimated by the committee of Lloyd's.

12 years.		Teak, British Oak, Mora, Greenheart, Iron-bark, Saul.
10	,,	Bay Mahogany, Cedar (*Juniperus Virginiana.*)
9	,,	European Continental Oak, Chestnut, Blue-gum, Stringy-bark (*Eucalyptus gigantea.*)
8	,,	North American White Oak, North American Chestnut.
7	,,	Larch, Hackmatack, Pitch Pine, English Ash.
6	,,	Cowrie, American Rock Elm.

5 years. Red Pine, Grey Elm, Black Birch, Spruce Fir, English Beech.
4 „ Hemlock Pine (North American.)

311. **Preservation of Timber.**—Amongst the most efficient means of preserving timber, are good seasoning and free circulation of air.

Protection against moisture is afforded by oil-paint, provided that the timber is perfectly dry when first painted, and that the paint is renewed from time to time. A coating of pitch or tar may be used for the same purpose, applied hot in thin layers.

Protection against the dry rot may be obtained by saturating the timber with solutions of particular metallic salts. For this purpose Chapman employed copperas (*sulphate of iron*); Mr. Kyan, corrosive sublimate (*bichloride of mercury*); Sir William Burnett, *chloride of zinc*. All these salts preserve the timber so long as they remain in its pores; but it would seem that they are gradually removed by the long-continued action of water. In planting poles it is advisable to fill the hole with cement or beton, terminated above, so as to let water drain off.

Dr. Boucherie employs a solution of *sulphate of copper* in about one hundred times its weight of water. The solution, being contained in a tank about 30 or 40 feet above the level of the log, descends through a flexible tube to a cap fixed on one end of the log, whence it is forced by the pressure of the column of fluid above it through the tubes of the vascular tissue, driving out the sap before it at the other end of the log, until the tubes are cleared of sap and filled with the solution instead.

Timber is protected not only against wet rot and dry rot, but against white ants and sea-worms, by Mr. Bethell's process of saturation with the liquid called commercially "*creosote*," which is a kind of pitch oil. This is effected by first exhausting the air and moisture from the pores of the timber in an air-tight vessel, in which a partial vacuum is kept up for a few hours, and then forcing the creosote into these pores by a pressure of about 150 lbs. on the square inch, which is kept up for some days. The timber absorbs from a *ninth* to a *twelfth* of its weight of the oil. (See p. 453.)

312. **Strength of Timber.**—Amongst different specimens of timber of the same species, those which are most dense in the dry state are in general also the strongest.

Tables of the average results of the most trustworthy experiments on the strength of different kinds of timber strained in various ways are given at the end of the volume; and a supplementary table containing some additional results, at the end of this section, p. 452. As to the strength of timber posts, see Article 158, p. 238.

The following are some general remarks as to the different ways in which the strength of timber is exerted:—

I. The TENACITY *along the grain*, depending, as it does, on the tenacity of the fibres of the vascular tissue, is on the whole greatest in those kinds and pieces of wood in which these fibres are straightest and most distinctly marked. It is not materially affected by temporary wetness of the timber, but is diminished by long-continued saturation with water, and by steaming and boiling.

The *tenacity across the grain*, depending chiefly on the lateral adhesion of the fibres, is always considerably less than the tenacity along the grain, and is diminished by wetness and increased by dryness. Very few exact experiments have been made upon it. Its smallness in pine-wood as compared with leaf-wood forms a marked distinction between those two classes of timber, the proportion which it bears to the tenacity along the grain having been found to be, by some experiments,

In pine-wood, from 1-20th to 1-10th.
In leaf-wood, from 1-6th to 1-4th, and upwards.

II. The RESISTANCE TO SHEARING, by sliding of the fibres on each other, is the same, or nearly the same, with the tenacity across the grain. As to *shearing across the grain*, see Article 322, p. 460.

III. The RESISTANCE TO CRUSHING a[illegible] the grain, depending, as it does, on the resistance of the fibres to being crippled or "upset," and split asunder, is greatest when their lateral adhesion is greatest, and has been found by Mr. Hodgkinson to be nearly twice as great for dry timber as for the same timber in the green state. In most kinds of timber, when dry, it ranges from one-half to two-thirds of the tenacity (p. 236).

Experiments have been made on the crushing of timber across the grain, which takes place by a sort of shearing; but they have not led to any precise result, except that the timber is both more compressible and weaker against a transverse than against a longitudinal pressure; and consequently, that intense transverse compression of pieces of timber ought to be avoided.

IV The MODULUS OF RUPTURE of timber, which expresses its resistance to cross-breaking, is usually somewhat less than its tenacity, but seldom much less. (See Article 162, p. 252.)

V. The FACTOR OF SAFETY, in various actual structures of carpentry, ranges from 4 to 14, and is on an average about 10.

When large sized pieces are tested, the strengths are found to be much below these obtained from small test pieces. See Lanza, *Applied Mechanics*.

SUPPLEMENTARY TABLE OF PROPERTIES OF TIMBER GROWN IN CEYLON; SELECTED AND COMPUTED FROM A TABLE OF THE PROPERTIES OF NINETY-SIX KINDS OF TIMBER BY MODLIAR ADRIAN MENDIS.

TIMBER.	Modulus of Elasticity in lbs. on the Square Inch.	Modulus of Rupture in lbs. on the Square Inch.	Weight of a Cubic Foot in lbs.
Aludel (*Artocarpus pubescens*),...	1,850,000	12,800	51
Burute (*Chloroxylon Swietenia*),	2,700,000	18,800	55
Caha Milile (*Vitex altissima?*),...	2,000,000	13,900	56
Caluvere. See "Ebony."			
Cos (*Artocarpus integrifolia*),......	1,810,000	11,000	42
Ebony or Caluvere (*Diospyros Ebenus*),....................	1,360,000	13,000	71
Gal or Hal Mendora (*Vateria sp. —?*)........................	1,330,000	13,300	57
Hal Milile (*Berrya Ammonilla*),	970,000	15,200	48
Ironwood. See "Naw."			
Jack. See "Cos."			
Mee (*Bassia longifolia*),.	1,880,000	13,000	61
Mecan Milile (*Vitex altissima*),...	2,040,000	14,200	56
Naw (*Mesua Nagaha*), ...	2,580,000	17,900	72
Palmira. See "Tal."			
Paloo (*Mimusops hexandra*),......	2,430,000	18,900	68
Satinwood. See "Burute."			
Soriya (*Thespesia populea*),.......	2,610,000	12,700	42
Tal (*Borassus flabelliformis*),......	2,810,000	14,700	65
Teak (*Tectona grandis*),...........	2,800,000	14,600	55

ADDITIONAL DATA FROM THE EXPERIMENTS OF CAPTAIN FOWKE, R.E., CAPTAIN MAYNE, R.E., AND MODLIAR MENDIS.

Teak from Johore (Malay Peninsula),		19,400	
Teak from Cochin-China,.........	1,990,000	12,100	44
Teak from Moulmein,.............	1,900,000	11,520	42
Iron-bark (*Eucalyptus—?*) from Australia,....................	964,000	24,400	64
Iron-bark, rough-leaved,...... ...	1,157,000	22,500	64
Jarrah. See "Australian Mahogany," in Tables at end of volume.			
Stringy-bark (*Eucalyptus gigantea*) from Australia,......	1,709,000	13,000	54

312A. **Preservation of Timber** for railway sleepers, &c., from decay is attempted by various processes. Kyanising consists in the immersion of the timber in a solution of corrosive sublimate, in the proportion of 1 part of sublimate to 100 parts of water, the wood being left in the solution for two days and upwards, according to thickness. In another process chloride of zinc is used. Creosoting consists in first placing the timber in a cylinder and applying superheated steam, and thereafter establishing a vacuum in the cylinder; by these operations the sap is withdrawn, when a "dead oil," containing creosote heated to about 160°, is forced at a pressure of about 20 lbs. into the cylinder, and fills up the space from which the sap has been removed. This process, however, varies with the condition of the wood when used.*

SECTION II.—*Of Joints and Fastenings in Carpentry.*

313. **Classification and General Principles.**—The *joints*, or surfaces at which the pieces of timber in a frame of carpentry touch each other, and the *fastenings* which connect those pieces together, are of various kinds, according to the relative positions of the pieces, and the forces which they exert on each other. *Joints* have been classed by Robison and Tredgold; and those authors are very nearly followed in the following classification, which will be the better understood by referring to the previous portion of this work which relates to framework in general, viz.:— Part II., Chapter I., Section IV., Articles 111 to 122, pp. 173 to 185:—

I. Joints for lengthening ties.
II. Joints for lengthening struts.
III. Joints for lengthening beams.
IV. Joints for supporting beams on beams.
V. Joints for supporting beams on posts.
VI. Joints for connecting struts with ties.

Fastenings may be classed as follows:—

I. Pins, including treenails, nails, spikes, screws, and bolts; being fastenings which are exposed principally to shearing and bending stress.

* For further information on the preservation of timber, see *Trans. American Society of Civil Engineers*, vol. xi.

See *Trans. Inst. Engineers and Shipbuilders in Scotland*, vol. xxv.

II. Straps and tie-bars, including iron stirrups and suspending-rods, being fastenings which are exposed principally to tension.

III. Sockets.

In designing and executing all kinds of joints and fastenings, the following general principles are to be adhered to as closely as may be practicable:—

I. To cut the joints and arrange the fastenings so as to weaken the pieces of timber that they connect as little as possible.

II. To place each abutting surface in a joint as nearly as possible perpendicular to the pressure which it has to transmit.

III. To proportion the area of each such surface to the pressure which it has to bear, so that the timber may be safe against injury under the heaviest load which occurs in practice; and to form and fit every pair of such surfaces accurately, in order to distribute the stress uniformly.

IV. To proportion the fastenings, so that they may be of equal strength with the pieces which they connect.

V. To place the fastenings in each piece of timber so that there shall be sufficient resistance to the giving way of the joint by the fastenings shearing or crushing their way through the timber.

314. **Lengthening Ties** is performed by *fishing* or by *scarfing*. In a fished joint the two pieces of the tie abut end to end, and are connected together by means of "fish-pieces" of wood or iron which are bolted to them; in a scarfed joint the ends of the two

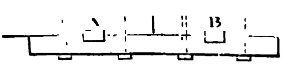


Fig. 187.

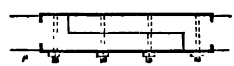


Fig. 188.

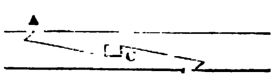


Fig. 189.

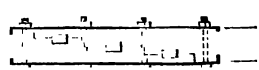


Fig. 190.

pieces of the tie overlap each other. Fig. 187 is a fished joint; figs. 188, 189, and 190 are called scarfs; though in figs. 188 and 190 the ties are in fact fished with iron as well as scarfed.

In a *plain fished joint* the fish-pieces have plane surfaces next the tie, so that the connection between them and the tie for the transmission of tension depends wholly on the strength of the bolts, together with the friction which they may cause by pressing the fish-pieces against the sides of the tie. The tie is only weakened so far as its effective sectional area is diminished by the bolt-holes. The joint sectional area of the fish-pieces should be equal to that of the ties. The joint sectional area of the bolts should be at least *one-*

fifth of that of the timber left after cutting the bolt-holes; and the bolts should be square rather than round. The bolt-holes should be so distributed, and placed at such distances from the ends of the two parts of the tie, that the joint area of both sides of the layer of fibres, which must be sheared out of one piece of the tie before the bolts can be torn out of its end, shall be as much greater than the effective area of the tie as the tenacity of the wood is greater than its resistance to shearing; as to which proportion, see Article 312, p. 450. The same rule regulates the places of the bolt-holes in the fish-pieces.

The fish-pieces and the parts of the tie may also be connected by *indents*, as at the upper side of fig. 187, or by *joggles* or *keys*, as at the lower side of the same figure. In either case the effective area of the tie is reduced by the cutting of the indents or of the key-seats, at A and B. The area of abutting surface of the indents, or of the key-seats, should be sufficient to resist safely the greatest force to be exerted along the tie; and their distances from the ends of the fish-pieces and of the parts of the tie should be sufficient to resist safely the tendency of the same force to shear off two layers of fibres.

A timber tie may be fished with plates of iron, due regard being paid to the greater tenacity of the iron in fixing the proportions of the parts, and the iron fish-plates may be indented into the wood. Fig 188 represents a joint in which the parts of the timber tie are scarfed together, and at the same time fished with iron plates, which are indented into the wood at the ends.

Fig. 189 represents a scarfed joint for a tie, which will hold without the aid of bolts or straps. At C is a key or joggle of some hard kind of wood, which is wedged in so as to tighten the joint moderately. The depth of the key is one-third of the depth of the beam. It is evident that this joint, as shown in the figure, has only one-third of the strength of the solid timber tie; but its strength may be considerably increased by bolting on iron fish-plates at A and B.

Fig. 190 shows a scarfed joint with several keys, which should all be driven equally tight. It is also fished with iron plates, indented into the wood at the ends.

The following practical rules are given by Tredgold for the proportion which the length of a scarf (between A and B in each of the figures) should bear to the depth of the tie:—

	Without Bolts.	With Bolts.	With Bolts and Indents.
Leaf-wood (as Oak, Ash, or Elm),....	6	3	2
Pine-wood,..............................	12	6	4

315. **Lengthening Struts.**—At each joint in a post, pillar, or other strut, the two pieces should abut against each other at a plane surface, perpendicular to the direction of the thrust; and to keep them steady they may either be fished on all four sides, or have their abutting ends enclosed in an iron socket made to fit them. Joints in struts ought if possible to be stayed laterally. (As to the strength of timber struts, see Article 158, p. 238).

316. **Lengthening Beams** may be performed either by fishing or by scarfing; and in either case the joints should as far as practicable be placed where the bending moment is small. The construction of the joints should be the same with that of joints for lengthening ties, with the following qualifications:—

I. At the compressed side of the beam, its two pieces should have a square abutment against each other; hence oblique surfaces, such as those in fig. 189, are to be avoided.

II. The surfaces of the scarf ought to be parallel to the direction of the load; (that is to say, in general, vertical: so that in figs. 188 and 190, the plane of the paper shall represent a horizontal plane); for it was found, in experiments by Colonel Beaufoy, that a scarfed beam was stronger with the scarf "up and down" than "flatwise." (See Barlow *On the Strength of Timber*, Article 71.)

317. **Notching Beams.**—When a joist or cross-beam has to be supported on a girder or main beam, the method which least impairs the strength of the main beam is simply to place the cross-beam above it; a shallow notch being cut on the lower side of the cross-beam, so as to fit the main beam.

318. **Mortising Beams—Shouldered Tenon.**—When the space is not sufficient to admit of placing the cross-beam above the main beam, the connection may be made by means of a *mortise and tenon joint;* the *tenon* being a projection from the end of the cross-beam, and the *mortise*, a cavity in the side of the main beam, cut so as exactly to fit the tenon. The tenon may be fixed in its place by means of a pin, or of a screw. It is evident that in order to weaken the main beam as little as possible, the mortise should be cut at the middle of its depth, so that the centre of the mortise may be at the neutral axis of the beam.

To find, in what proportion a beam is weakened by a plain rectangular mortise cut in the position above prescribed, let h be the depth and b the breadth of the beam, h' the depth of the mortise, and b' the distance to which it penetrates into the beam; then the beam is weakened in the following ratio:—

$$b\,h^3 - b'\,h'^3 : b\,h^3. \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(1.)$$

(See Article 162, pp. 249 to 253.)

To keep a cross-beam steady in its proper position, a tenon

requires length; to bear its share of the load, it requires depth; but a tenon at once long and deep would too much weaken the main beam. To avoid this difficulty the *shouldered tenon* is used, as shown in fig. 191. A is a cross-section of a main beam; B is one end of a cross-beam. C is the shoulder, which bears the load of that end of the cross-beam, and penetrates into the side of the main beam for a distance of one-sixth of the depth of the cross-beam or thereabouts; the depth of the shoulder below the upper side of the cross-beam is about two-thirds or three-fourths of the total depth of that beam. D is the tenon proper, whose depth is only one-sixth of that of the cross-beam, while its length is about double of its own depth. Its use is to give the joint sufficient hold, so that there shall be no risk of the shoulder being dislodged from its place in the mortise.

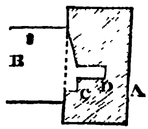


Fig. 191.

Mortises cut by hand are always rectangular. Those cut by machinery are made by a boring tool, so that although their longest sides are plane, their ends are semicylindrical; and tenons to fit them must be cut of the same shape.

319. **Post and Beam Joints.**—To support the end of a horizontal beam at one side of a post, a shouldered mortise-and-tenon joint is to be used. The shoulder should be like that on the end of the cross-beam in fig. 191; but the long tenon should be *on edge*, or have its narrowest dimension horizontal, in order that the mortise for it may weaken the post as little as possible.

When the beam is to rest on the top of the post, the joint may be secured simply by means of a small tenon in the centre of the top of the post fitting into a mortise in the under side of the beam; but there are other methods, two of which are shown in fig. 192. B B is the beam. A is a post, the top of which is fitted into a shallow rectangular notch in the under side of the beam. That notch does not extend completely across the beam, but is divided into two parts by a *bridle*, of about one-fifth of the breadth of the beam, which is left uncut in the middle of the notch. To receive the bridle, a groove of the same breadth is cut in the middle of the top of the post, as indicated by the dotted line C D. The post E is also fitted into a notch-and-bridle joint F G, the only difference being that the figure of the notch in the under side of the beam is an obtuse angled triangle instead of a rectangle. This last form is recommended by Tredgold. He also recommends a joint of the same class, in which the notch in the under side of the beam has the figure of a circular arc; but from

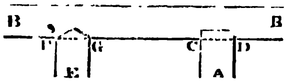


Fig. 192.

the experiments of Mr. Hodgkinson on the strength of flat-ended and round-ended pillars, it must be inferred that this construction would weaken the post. (Article 158, pp. 236 to 238.)

The same joints are applicable to the case in which a post is supported on a beam.

320. **Strut-and-Tie Joints.**—A strut and a tie meeting at an oblique angle are to be connected by means of a shoulder on the end of the strut, fitting into a notch in the side of the tie, to transmit the pressure, and of a tenon on the strut fitting into a mortise in the tie, or a bridle on the tie fitting into a groove in the shoulder of the strut, to keep the joint steady. Such joints are exemplified in figs. 193 and 194, in each of which B represents a tie-beam and A the foot of a strut or rafter. C D is the *shoulder* of the rafter, fitting into a notch in the tie-beam, and having a plane surface, which in fig. 193 has a depth equal to half of the depth of the rafter, and bisects the obtuse angle between the directions of the tie-beam and rafter; while in fig. 194 it is perpendicular to

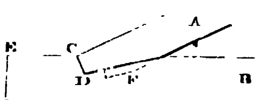


Fig. 193.

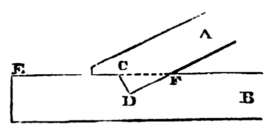


Fig. 194.

the length of the rafter, and of somewhat more than half its depth. In fig. 193 the dotted lines at F represent a *tenon and mortise*, whose breadth is one-fifth of that of the rafter. In fig. 194, the dotted line C F shows the upper surface of a *bridle*, left uncut in the middle of the breadth of the notch C D F in the tie-beam, and fitting into a groove in the shoulder of the rafter. The breadth of the bridle is one-fifth of the breadth of the tie-beam.

In making each of those joints, care must be taken that the length of the fibres left between the notch C D and the end E of the tie-beam is sufficient to resist safely the tendency of the longitudinal component of the thrust against the notch to shear them off; that is to say, let H be that component of the thrust of the rafter, b the breadth of the tie-beam in inches, l the distance in inches from the notch to the end of the tie-beam, f' the resistance of the wood to shearing, s a factor of safety; then

$$l = \frac{s\,H}{f'\,b}; \qquad (1.)$$

According to Tredgold, 4 is a sufficient value for s in this case; and hence, taking f' at 600 lbs. per square inch for fir, and 2,300 lbs. per square inch for oak, we have

$$\text{For oak, } l = \frac{H}{575\,b}\text{; for fir, } l = \frac{H}{150\,b}. \qquad (1\text{ A.})$$

These joints may be made more secure by binding the rafter and tie together with a bolt or a strap, in a direction making as acute an angle with the tie as is practicable. The chief object of this is to hold the rafter in its place in case the end of the tie should give way. (See fig. 197, p. 462.)

321. **Suspending Pieces** in frames of carpentry are called by the very inappropriate names of *king-posts* and *queen posts*, a king-post being a single suspending piece in the centre of a frame, and queen-posts, suspending pieces in other positions. A suspending piece hangs from the point of junction of two struts or rafters, and supports at its lower end either a beam or the ends of one or more struts.

A strut or rafter may be connected with a suspending piece by abutting against a notch cut in its side, or against a shoulder formed by an enlargement at the end of the suspending piece; and in either case the distance of the notch or shoulder from the end of the piece is to be determined by the formulæ of the preceding article. When a single suspending piece supports a beam at its lower end they are connected by means of an iron stirrup.

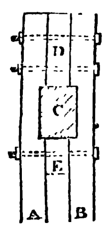


Fig. 195.

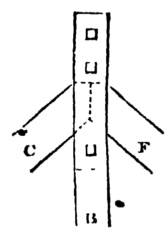


Fig. 196.

A better method is to make suspending pieces in pairs, so that the rafters from which they hang may abut between them directly against each other, as shown by the cross-section fig. 195, and the side view fig. 196. C and F are the ends of a pair of rafters abutting against each other; A and B the upper ends of a pair of suspending pieces, notched upon the rafters, and bolted to each other through the blocks or filling-pieces D and E. If these figures be turned upside down they will represent the lower ends of a pair of suspending-pieces, forming a wooden stirrup for the support of a beam, or of the ends of a pair of struts, as the case may be.

322. **Pins—Treenails.**—Wooden pins, as fastenings for joints, when of large diameter, are known as *treenails*. Experiments have been made on their resistance to a cross strain by Mr. Parsons, for the details of which, see Murray *On Ship-building;* the results may be summed up with sufficient exactness for practical purposes by saying—

I. That the ultimate resistance of English oak treenails to a

shearing stress across the grain is about 4,000 lbs. per square inch of section.

II. That in order to realize that strength, the planks connected by the treenails should have a thickness equal to about three times the diameter of the treenails.

323. **Nails and Spikes.**—Where nails are exposed to any considerable strain those made by hand should be used, as they are stronger than those made by machinery.

The weight in lbs. of a thousand of the "flooring brads" commonly used in carpentry may be roughly computed by taking *twice the square of their length in inches.*

The nails or spikes used for fastening planks to beams are usually of a length equal to from twice to twice and a-half the thickness of the planks.

The following are the results, as stated by Tredgold, of experiments by Bevan on the force required to draw nails of different sizes out of *Dry Christiania Deal*, into which they had been driven to different depths *across the grain*:—

Kind of Nails.	Length. Inches.	No. to the Lb.	Inches driven	Force to draw. Lbs.
Sprigs, ...	0·44	4,560	0·4	22
,, ...	0·53	3,200	0·44	37
Threepenny brads,	1·25	618	0·50	58
Cast iron nails, ...	1·00	380	0·50	72
Fivepenny nails,	2·00	139	1·50	320
Sixpenny nails, ...	2·50	73	1·00	187
,, ...	2·50	73	1·50	327
,, ...	2·50	73	2·00	530

So far as these results can be expressed by a general law, they seem to indicate that the force required to draw a nail, driven across the grain of a given sort of wood, varies nearly as the *cube of the square root of the depth to which it is driven;* and that it increases with the diameter of the nail, but in a manner which has not yet been expressed by a mathematical law.

The following are the results of Bevan's experiments on the force required to draw a "sixpenny nail" of 73 to the lb., which had been driven one inch into different sorts of timber:—

Deal, across the grain, ...	187	lbs. (as above.)
Oak, ,, ...	507	,,
Elm, ,, ...	327	,,
Beech, ,, ...	667	,,
Green Sycamore, ,, ...	312	,,
Deal, endwise, ...	87	,,
Elm, ,, ...	257	,,

The following were the forces required to draw asunder a pair of planks joined by *two nails* of 73 to the lb. :—

Deal $\frac{7}{8}$ inch thick, ...	712 lb.
Oak 1 inch thick, ...	1009 „
Ash 1 inch thick, ...	1420 „

324. **Screws.**—The holding power of screw-nails, or "wood-screws," is probably proportional nearly to the product of the diameter of the screw, and of the depth to which it is screwed into the wood. The following are the results of Bevan's experiments, quoted by Tredgold, on the force required to draw screws out of planks of *half-an-inch thick*, the screws being 0·22 inch in diameter over all, and 0·035 inch in depth of thread, with 12 threads to the inch.

Beech, ...	460 to 990 lbs.
Ash, ...	790 lbs.
Oak, ...	760 „
Mahogany, ...	770 „
Elm, ...	665 „
Sycamore, ...	830 „

325. **Bolts—Washers.**—The rules for proportioning bolts which have to withstand a shearing stress in carpentry have already been stated in Article 314, p. 455.

The sides of a piece of timber should always be protected against the crushing action of the head and nut of a bolt by means of flat rings called "*washers*;" the area of each washer being at least as many times greater than the sectional area of the bolt as the tenacity of the bolt is greater than the resistance of the timber to crushing; that is to say, for fir the diameter of the washer may be made about $3\frac{1}{2}$ times the diameter of the bolt, and for oak about $2\frac{1}{2}$ times.

When a bolt is oblique to the direction of the beam that it traverses, the timber may either have a notch cut in it with a surface perpendicular to the bolt, to bear the pressure of the washer, or it may be notched to receive a bevelled washer of cast iron, one of whose surfaces fits the notch in the wood, while another being perpendicular to the axis of the bolt, bears the pressure of the nut or head, as the case may be.

The screws of bolts are usually made of the following proportions, or nearly so: the depth of the thread one-tenth, and the pitch one-fifth of the internal diameter. A bolt which has to be often removed may be made fast by having a slot or oblong hole in one of its ends, through which a key or wedge is driven.

326. **Iron Straps** are used nearly in the same manner with bolts, to bind pieces of timber together. They have the advantage of not requiring so much of the timber to be cut away as bolts do.

According to the usual proportions of straps the breadth ranges from four times to eight times the thickness. When a strap has eyes in its ends, for bolting them to the sides of a beam, it ought to be either broadened or thickened round each eye, so that the sectional area of the iron may be at least as great at the sides of the eye as in other parts of the strap. When a strap is to embrace completely a piece or pieces of timber, it may, when practicable, be welded into a rectangular hoop, and driven on from one end of the timber; but when that is impracticable or inconvenient, it must be made with screws on its ends, of the same sectional area with its flat part, upon which screws a cross-piece is to be made fast with nuts.

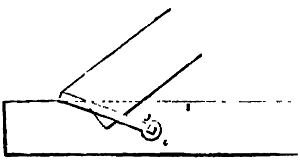


Fig. 197.

327. **A Stirrup** is a strap which supports a beam, or sustains the thrust of one end of a strut. If the tie or suspending piece is of wood, the ends of the stirrup are bolted through it; if of iron, the stirrup and tie, or suspending rod, are usually welded into one piece.

328. **Iron Tie-Rods** may be used instead of timber ties and suspending pieces in all these parts of a frame of carpentry in which tension alone is to be borne, and is not combined with a bending action, nor alternated with thrust. They may be connected with the timber pieces of the frame by means of screws and nuts, eyes and bolts, slots and wedges, stirrups or sockets; and they should be capable of being tightened when required, by means of screws or of wedges. Care must be taken that the points of attachment of the ends of a long iron tie-rod are free to change their distance from each other to an extent sufficient to allow of the changes of length of the rod which are produced by changes of temperature, at the rate of about

·0012 of the length of the rod, for 180° of change of temperature on Fahrenheit's scale.

329. **Iron Sockets,** made to fit the ends of pieces of timber, furnish a convenient means of making various joints in framework, especially at points where struts meet each other, or have to be connected with tie-rods. If thrust alone is to be borne by the socket, cast iron is the most convenient material; if any considerable tension is to be borne, strong wrought iron plates are best.

330. **Protection of Iron Fastenings.**—The iron fastenings of timber, especially if in contact with oak, rust very rapidly unless properly protected. Amongst the most efficient means of protection are the following:—

I. Boiling in coal-tar, especially if the pieces of iron have first been heated to the temperature of melting lead.

II. Heating the pieces of iron to the temperature of melting lead, and smearing their surfaces, while hot, with cold linseed oil, which dries and forms a sort of varnish. This is recommended by Smeaton.

III. Painting with oil-paint, which must be renewed from time to time. The linseed oil process is a good preparation for painting.

IV. Coating with zinc, commonly called galvanizing. This is efficient, provided it is not exposed to acids capable of dissolving the zinc; but it is destroyed by sulphuric acid in the atmosphere of places where much coal is burned, and by muriatic acid in the neighbourhood of the sea. (See p. 799.)

Section III.—*Of Timber Built Beams and Ribs.*

331. **Joggled and Indented Built Beams.**—In fig. 198 two pieces of timber are built into one beam of double the depth of either, by the aid of hardwood *keys* or *joggles*, which resist the shearing stress at the surface of junction, and of vertical bolts in the spaces between the keys. It is obvious that no key nor bolt should be put at the middle of the span; because in general there is no shearing stress there; and also because the bending moment is in general a maximum there, and it is desirable to weaken the cross-section as little as possible. The grain of the keys should run vertically. According to Tredgold, the joint depth of all the keys should amount to *once and a-third* the total depth of the beam, and the breadth of each key should be twice its depth.

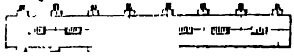


Fig. 198.

Considering that the stress at the neutral surface is equivalent to thrust in a direction sloping at 45°, combined with tension in a direction sloping at 45° the opposite way (see Article 162, p. 250), it would seem that the best position for the keys would be that shown in fig. 199, their fibres being made to slope in the direction of the thrust, and the bolts being made to slope in the direction of the tension. This, however, so far as I know, has never yet been tried.

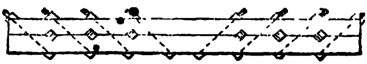


Fig. 199.

In fig. 200 the two pieces of which the beam is built are indented into each other, a sacrifice of depth being thus incurred equal to the depth of an indent. The abutting surfaces of the indents face outwards in the upper piece, and inwards in the

lower, so as to resist the tendency to slide. According to experiments by Duhamel, the joint depth of the indents should amount to *two-thirds* of the total depth of the beam. The beam in the figure is slightly tapered from the middle towards the ends, in order that the hoops which are used to bind it may be put on at the ends and driven tight with a mallet.

Fig. 200.

When a beam is built of several pieces in length as well as in depth, they should break joint with each other. The lower layer should be scarfed or fished like a tie (Article 314, p. 454), and the upper layer should have plain butt joints. The upper layer of a built beam is sometimes made of hardwood, and the lower layer of fir, in order to take advantage of the resistance of the former to crushing and the tenacity of the latter.

332. **Bent Ribs** are sometimes obtained from naturally bent pieces of timber, called "knees."

Naturally straight pieces of timber may be permanently bent by steaming them until the wood is softened, and while in that condition bending them by combinations of screws, and keeping them bent until they dry and stiffen. By this process there is a risk of injuring the tenacity of the fibres at the convex side of the piece, unless they are prevented from stretching by the following contrivance (see fig. 201):—A A is the piece of wood to be bent. Its ends abut against the bent parts of a strip of boiler-plate B B, which has two eyes C C, that are drawn together by a pair of tightening-screws at D till the required curvature is produced. The whole of the fibres of the timber are compressed, and none of them have their tenacity injured; and it is found by experiment that bent ribs made in this way are as strong as natural knees.

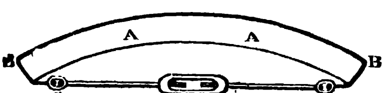


Fig. 201.

333. **Built Ribs** are best made by a method invented by Philibert de l'Orme, and represented in figs. 202, 203, and 204. Fig. 202 is a side view, and fig. 203 a plan of a rib made of several layers of planks *set on edge*, breaking joint with

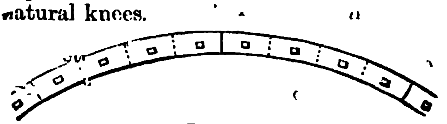


Fig. 202.

Fig. 203.

each other (as the plan shows), and connected together by square bolts or wedges.

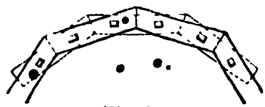


Fig. 204.

In fig. 202 the edges of the planks are supposed either to have been originally curved, or have had the corners smoothed off: in fig. 204 it is shown how they may be used with straight edges.

A built rib of this sort, properly constructed is nearly as strong as a solid rib of the same depth, and of a breadth less by the thickness of *one layer*.

334. **Laminated Ribs** are composed, as in fig. 205, of layers of plank *laid flatwise*, breaking joint and bolted together. They are easily made, and very often used in bridges and roofs; but the experiments of Ardant have shown that they are weaker than solid ribs of the same dimensions, nearly in the ratio of unity to the number of layers into which they are divided.

Fig. 205.

SECTION IV.—*Of Timber Frames and Trusses.*

335. **General Remarks on the Balance, Stability, and Strength of Timber Framing.**—The general principles of the balance and stability of frames and ribs of any material, already given in Part II., Chapter I., Section IV., pp. 173 to 203, and the general principles of the strength of materials, given in the same chapter, Section V., pp. 221 to 314, serve to solve all problems relating to the balance, stability, and strength of structures in carpentry. In the present section it will only be necessary to add some explanations of matters of detail in those particular cases which occur most frequently in practice. In fixing the transverse dimensions, or "*scantlings*," of the main pieces of timber which compose a structure of carpentry, made of good pine, fir, or oak, it is usual to limit the greatest intensity of the stress, whether compressive or tensile, to 1,000 *lbs. per square inch of section;* and when this is compared with the tenacity, resistance to crushing, and modulus of rupture, of those kinds of timber, it appears that the *factor of safety* ranges from 6 to 14, or thereabouts, and is on an average 10, as has been stated in Article 143, p. 222.

336. **Platforms** of timber consist of planks resting on beams. The beams upon which the planks rest may either be the main beams or *girders* of the structure, or they may be cross-beams or *joists*, supported by those girders. (Articles 317, 318, pp. 456,

457.) The former mode of construction is that which enables a given strength to be attained with the least expenditure of material and labour at the outset; but the latter, in most cases, is the more economical in the end; for although it causes a greater expenditure of material in joists than it saves by requiring thinner planking, the saving in the quantity of planking is productive of the greatest saving of expense; for the planking requires more frequent renewal than the joists.

It would be foreign to the purpose of this book to describe the various modes of constructing the floors of houses. The timber platforms with which the civil engineer is chiefly concerned are those of bridges and of foundations. The latter will be described further on.

The usual thickness of the planking for the platform of a bridge with joists is from 3 to 4 inches, the joists being placed at distances of from 2 feet to 4 feet from centre to centre. That thickness has been found by experience to be requisite in order to withstand the shocks, friction, and wear, to which the planking is subjected, and is in general much greater than is required for mere strength to support the greatest load with safety.

In bridges supporting railways where chairs are used, the joists are usually so arranged as to be directly under the chairs.

The breadth of the joists is from one-eighth to one-quarter of their distance apart; and their transverse dimensions are fixed with reference to the greatest load upon them and to the width which they span over between the girders. For timber bridges and platforms not carrying railways, that load, *in lbs. per square foot of platform*, is nearly as follows:—

Weight of a closely-packed crowd, estimated at	120 lbs.	per sq. ft.
Add for the planking and joists, say	30 ,,	,,
Gross load for a single wooden platform, ...	150 ,,	,,
If there is a broken stone or gravel roadway, add	100 ,,	,,
Making in all	250 ,,	,,

When the platform carries a railway, the scantling of each joist must be regulated by the fact, that the load on a pair of driving wheels of the heaviest engine used on the line may rest above a certain pair of points in the joist. Should the rails be either directly above the girders, or so nearly above them that this rule gives a less scantling than the former, the rule for platforms not carrying railways is to be followed.

The best mode, in general, of designing the joists, is to fix the ratio of the depth to the span with a view to stiffness, as ex-

plained in Article 170, p. 275, and then compute the breadth with a view to strength.

The following formulæ express the results of these rules algebraically.

CASE I. For platforms not carrying railways, let

B be the distance from centre to centre of the joists.
b, the breadth of a joist.
h, its depth.
l, the span from centre to centre of the girders; then the greatest moment of flexure is to be as follows:—

$$\left.\begin{array}{l} \dfrac{1{,}000\,b\,h^2}{6} = \dfrac{150\ \mathrm{B}\,l^2}{8 \times 144} \text{ for plank roadways;} \\ \text{or } \dfrac{250\ \mathrm{B}\,l^2}{8 \times 144} \text{ for broken stone roadways;} \end{array}\right\} \text{......(1.)}$$

and consequently,

$$\left.\begin{array}{l} \dfrac{b}{\mathrm{B}} = \dfrac{l^2}{1{,}280\,h^2} \text{ for plank roadways;} \\ \dfrac{b}{\mathrm{B}} = \dfrac{l^2}{768\,h^2} \text{ for broken stone roadways.} \end{array}\right\} \text{......(2.)}$$

CASE II. For a platform carrying a railway, in which one line of rails lies midway between a pair of girders, let

W be the load on a pair of driving wheels of the heaviest engine, in lbs
k, the gauge of the rails, *from centre to centre in inches;* then, l being also expressed in inches,

$$\frac{1{,}000\,b\,h^2}{6} = \frac{\mathrm{W}\,(l-k)}{4}; \text{ ....(3.)}$$

and therefore

$$b = \frac{3}{2{,}000} \cdot \frac{\mathrm{W}\,(l-k)}{h^2} \text{ ....(4.)}$$

Example.—Let $l = 90$ inches; $k = 60$ inches; $h = 12$ inches; W = 30,000 lb.; then $b = 9{\cdot}375$ inches.

As to the length and weight of the spikes to be used for nailing the planks to the joists, see Article 323, p. 460.

When a platform has both girders and joists, it may be stiffened against distortion by laying the planks diagonally. When separate diagonal braces are used for that purpose, their dimensions should be regulated by the horizontal shearing stress which the wind may

produce when blowing against the side of the structure, as calculated by the formula for F in Case VI. of the table, p. 246. The greatest intensity of the pressure of the wind hitherto observed in Britain is 55 lbs. on the square foot; in tropical climates it is said sometimes to reach double that amount. (See p. 222*b*.)

When there is no special reason for making a timber platform close-jointed, it is advisable to lay the planks with openings between them of from $\frac{1}{4}$ inch to $\frac{1}{2}$ inch in width, in order to let rain-water escape and air circulate.

337. **Roofs—Covering and Load.**—The parts of a roof may be distinguished into the *covering* and the *framework*. The extent of the covering of a roof is usually expressed in *squares* and *feet*, a *square* of roofing being 100 square feet. The following table shows the structure and weight in lbs. per square foot of the most usual kinds of covering for timber roofs, and their flattest ordinary slopes:*

Material.	Flattest Ordinary Slope.	Weight per Square Foot—Lbs.
Sheet copper, about ·022 of an inch thick,	4°	1·00
Sheet lead,	4°	7·00
Sheet zinc,	4°	1·25 to 1·625
Sheet iron, plain, $\frac{1}{16}$ inch thick,	4°	3·00
„ corrugated,	4°	3·40
Cast iron plates, $\frac{3}{8}$ inch thick,	4°	15·00
Slates,	30° to 22½	5·00 to 11·20
Tiles,	30° to 22½	6·50 to 17·80
Boarding, $\frac{3}{4}$ inch thick, (Weight of other thicknesses in proportion.)	22½°	2·50
Thatch,	45°	6·50
For the timbering of slated and tiled roofs, add per square foot,		from 5·50 to 6·50
For the pressure of the wind, according to Tredgold, there is to be taken into account an additional load per square foot of		40

Sheet copper is nailed on boards. Sheet lead, zinc, and iron, slates, and tiles, may be either nailed on laths or battens (which are slender pieces of timber of from 1 inch by 1½ inch to 1¼ inch

* The angles set down for the slopes of roofs in this table are all aliquot parts of a circumference; such angles being at once the most convenient in designing framework, and the most pleasing to the eye. (The latter fact appears to have been first pointed out by Mr. Hay in his *Theory of Beauty*.)

by 3 inches, or thereabouts, nailed across the rafters), or upon boarding of from ½ inch to ¾ inch thick. Sheet iron may be nailed or screwed directly to the rafters, and cast iron plates screwed or bolted to the *principal rafters* to be afterwards mentioned. Roofs in which the framework as well as the covering is of iron will be treated of in another chapte

The *steepest* ordinary declivity in Gothic roofs is 60°; but by the Metropolitan Building Act, 1855, the declivity of the roofs of buildings used for purposes of trade is limited to 47°.

338. **Rafters and Purlins** are those parts of the framework of a roof which lie immediately below the covering, so as to form with it a more or less sloping platform. In fig. 206, A A is one of the *common rafters*, which are placed from 1 foot to 2 feet apart from centre to centre, and are supported by the *purlins*, to which they are spiked or screwed. B is a cross-section of one of the purlins, which lie from 6 feet to 8 feet apart from centre, and are slightly notched where they cross the *principal rafters*. The side of the purlin which faces down the slope is supported by means of the block C, which is screwed to the principal rafter D D. The principal rafters form parts of a series of frames or *trusses*, which are placed at from 5 to 10 feet apart. In order to prevent the action of transverse loads on the principal rafters, they are to be supported below each point where the purlins cross them by struts, such as that of which the upper end is shown at E.

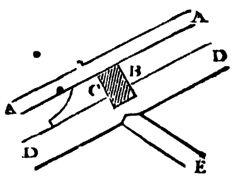


Fig. 206.

Diagonal Braces, to stiffen the roof and stay the trusses against upsetting sideways, may be framed either between the rafters or between the purlins. No precise rule can be given for their scantling; but they will in general be strong and stiff enough if each transverse dimension is made one-twentieth part of the unsupported length. When the roof is boarded, the same purpose may be answered by laying the boards diagonally.

339. **Roof-Trusses** are frames of the kinds already discussed in Articles 114 to 120, pp. 176 to 184, in which the principles that regulate the thrusts and tensions along the several pieces have been explained. In the present Article it is only necessary to state what particular cases of such frames are the most common in practice.

I. Triangular Truss.—Fig. 207 is a skeleton figure of the simplest form of truss, which is an isoceles triangle, B B being the tie-beam, and A and C equally inclined principal rafters. 2 and 3 are the points of support, 1 the ridge. D is a suspending-piece, which, when of wood, is called the *king-post*, and when of iron, the

king-bolt; it supports the weight of the middle half of the tie-beam,

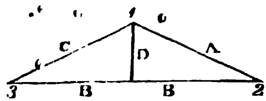


Fig. 207.

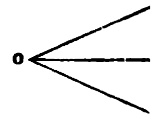


Fig. 208.

and of any floor or other load with which that beam may be loaded.

In the diagram, fig. 208, the vertical line C A represents the load on the point 1; that is, half the gross weight of the roof; O C and O A, parallel to the two rafters, represent the thrusts along them; and the horizontal line O B represents the tension along the tie-beam.

The algebraical expression of this is as follows:—

Let W be the gross weight of the truss, together with that of the division of the roof, of which it occupies the middle, and that of the floor, or other load supported by the tie-beam.

c, the half-span of the truss.
k, its rise.
H, the tension along the tie-beam.
T, the thrust along each of the rafters; then

$$H = \frac{W c}{4 k};\ T = \sqrt{\left(H^2 + \frac{W^2}{16}\right)} \ldots\ldots \quad (1.)$$

II. Trapezoidal Truss.—In fig. 209, B B B is the tie-beam, A and C two equally inclined principal rafters, F a horizontal rafter or *straining-piece.* D and E are suspending-pieces, to carry part of the weight of the tie-beam, and also that of the floor, which usually rests on the tie-beam between the points 5 and 6, together with its load.

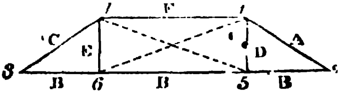


Fig. 209.

The same diagram of forces as in the former case, fig. 208, applies to this case; it being understood that C B = B A represent the loads on the points 1 and 4 respectively; that is, on each of those points, *one quarter of* the weight of the roof and truss, and *half* the weight of the floor between the points 5 and 6. The horizontal line O B represents at once the tension along the tie-beam, and the thrust along the straining-piece F.

The part of the roof above the straining piece F may either be flat, or may be supported by a small triangular ***secondary truss***

(see Article 121, p. 184), similar to fig. 207, and resting on the points 1 and 4. The straining-piece F of the principal truss may be made to act also as the *tie-beam* of the secondary truss; in which case the thrust along it will be the *excess of the horizontal stress* H *in the principal truss above that in the secondary truss.*

III. Secondary Trussing under Principal Rafters.—The direct support of the points where the purlins cross the rafters, already mentioned in Article 338, p. 469, is effected by means of a system of secondary trussing, of which fig. 210 may be taken as an example. That figure represents a truss in which the main tie and the suspending-pieces are all iron rods; but it is applicable also to the case in which either some or all of those pieces are of timber. (*A. M.*, 159.)

Let W be the weight of the roof distributed over the points 3, 4, 6, 1, 8, 10, 2, so that *one-twelfth* rests directly on each of the points of support 2 and 3, and *one-sixth* on each of the five

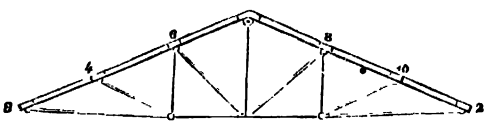


Fig. 210.

intermediate points; 2 3 is the great tie-rod; 1 7, 6 5, 8 9, suspending-rods; 7 6, 7 8, 5 4, 9 10, struts.

(1.) *Primary Truss* 1 2 3.—The load at 1, as before, is to be taken as $= \frac{1}{2}$ W, and the stresses found by equation 1 of this article.

(2.) *Secondary Trusses* 7 6 3, 7 8 2.—The load at 6 is to be held to consist of one-half of the load between 6 and 1, and one-half of the load between 6 and 3; that is, one-half of the load between 1 and 3, or $\frac{1}{4}$ W. The trusses are triangular, each consisting of two struts and a tie, and the stresses are to be found as in Article 115, p. 177; that is to say, let H′ denote the horizontal stress in each of these secondary trusses; T′ the thrust along the rafters between 6 and 3, and between 8 and 2, due to their places in those trusses; and S′ the thrust along the struts 67 and 87; then

$$H' = \frac{W}{12}\frac{c}{k};\ T' = \sqrt{\left(H'^2 + \frac{W^2}{144}\right)};\ S' = \sqrt{\left(H'^2 + \frac{W^2}{36}\right)}.\quad (2.)$$

The suspension-rod 1 7 supports two-thirds of the load on 7 6 3, and two-thirds of the load on 7 8 2; that is, $\frac{2}{3} \cdot \frac{2}{4} \cdot W = \frac{1}{3}$ W; and

this, together with $\frac{1}{6}$ W, which rests *directly* on 1, makes up the load of $\frac{1}{3}$ W, already mentioned.

(3.) *Smaller Secondary Trusses* 3 4 5, 9 10 2.—Each of the points 4 and 10 sustains a load of $\frac{1}{6}$ W, from which the stresses on the bars of those smaller trusses can be determined as follows:—

$$H'' = \frac{W}{12}\frac{c}{k};\ T'' = S'' = \sqrt{\left(H''^2 + \frac{W^2}{144}\right)}. \text{.......}(3.)$$

One-half of the load on 4, that is, $\frac{1}{12}$ W, hangs by the suspension-rod 6 5; and this, together with $\frac{1}{6}$ W, which rests directly on 6, makes up the load of $\frac{1}{4}$ W on that point, formerly mentioned. The same remarks apply to the suspension-rod 8 9.

(4.) *Resultant Stresses.*—The pull between 5 and 9 is the sum of those due to the primary and larger secondary trusses; that between 5 and 3, and between 9 and 2, is the sum of the pulls due to the primary, larger, secondary, and smaller secondary trusses; that is to say,

$$H + H' = \frac{W}{3}\frac{c}{k};\ H + H' + H'' = \frac{5\ W}{12}\frac{c}{k}; \text{......}(4.)$$

The thrust on 1 6 is due to the primary truss alone; that on 6 4 to the primary and larger secondary truss; that on 4 3 to the primary, larger secondary, and smaller secondary trusses; and similarly for the divisions of the other rafter.

(5.) *General Case.*—Suppose that instead of only three divisions, there are n divisions in each of the rafters 1 3, 1 2, of fig. 78; so that besides the middle suspension-rod 1 7, there are $n - 2$ suspension-rods under each rafter, or $2n - 4$ in all; and $n - 1$ sloping-struts under each rafter, or $2n - 2$ in all. There will thus be $2n - 1$ centres of resistance; that is, the ridge-joint 1 and $n - 1$ on each rafter; and the load *directly supported* on each of these points will be $\frac{W}{2n}$.

The total load on the ridge-joint 1, will be as before, $\frac{W}{2}$; that is to say, $\frac{W}{2n}$ directly supported, and $\frac{W}{2}\left(1 - \frac{1}{n}\right)$ hung by the middle suspension-rod.

The total load on the upper joint of any secondary truss, distant from the ridge-joint by m divisions of the rafter, will be, $\frac{n - m + 1}{4n}$ W; that is to say, $\frac{W}{2n}$ directly supported, and $\frac{n - m - 1}{4n}$ W hung by a suspension-rod.

The stresses on the struts and tie of each truss, primary and secondary, being determined as in Article 115, are to be combined as in the preceding examples.

The following formulæ give the horizontal stress H_m, the thrust along the rafter T_m, and the thrust along the strut S_m, in that secondary truss which has its highest point at m divisions of the rafter from the ridge-joint:—

$$H_m = \frac{W c}{4 n k}; \text{ (being the same for each secondary truss);...(5.)}$$

$$T_m = \sqrt{\left(H_m^2 + \frac{W^2}{16 n^2}\right)} = \frac{W}{4 n} \sqrt{\left(\frac{c^2}{k^2} + 1\right)};$$

(also the same for each secondary truss.)

$$S_m = \sqrt{\left\{ H_m^2 + \frac{W^2}{16 n^2} \cdot (n - m)^2 \right\}} = \frac{W}{4 n} \sqrt{\left(\frac{c^2}{k^2} + (n - m)^2\right)}. \text{ (7.)}$$

It follows that the total tensions on the several divisions of the tie-rod and thrusts on the several divisions of the rafters, commencing at the divisions next the middle suspending-rod, are as follows $\left(\text{making } \frac{W c}{4 k} = H, \text{ and } \frac{W}{4} \sqrt{\left(\frac{c^2}{k^2} + 1\right)} = T, \text{ as in the equations 1}\right)$;

$$H \left(1 + \frac{1}{n}\right); H \left(1 + \frac{2}{n}\right); \text{ \&c., } \ldots \ldots H \cdot \frac{2 n - 1}{n}; \text{ (8.)}$$

$$T; T \left(1 + \frac{1}{n}\right); T \left(1 + \frac{2}{n}\right); \text{ \&c., } \ldots \ldots T \cdot \frac{2 n - 1}{n}. \text{ (9.)}$$

In timber roofs, instead of resisting the horizontal thrust of such struts as 4 5 and 9 10 by means of tie-rods, it is usual to make their lower ends abut against a horizontal strut or straining-piece laid on the top of the main tie-beam, and extending from 5 to 9; the object being to give transverse strength to the tie-beam. In that case the tension is uniform along the whole length of the tie-beam, being $H \cdot \frac{2 n - 1}{n}$.

IV. GOTHIC ROOF-TRUSSES belong to the class of "Open Polygonal Frames," already mentioned in Article 117, p. 179; and they exert oblique thrust against the walls or buttresses which support them. The framing is so designed as to make the horizontal component of that thrust as small as possible.

Fig. 211 may be taken as an example. In this truss B C and B′ C′ are horizontal ties, each extending over one-quarter of the span, and E F is a suspending-piece; all the other pieces are struts. The struts A C and A′ C′ are curved for the sake of architectural effect; their straight lines of resistance, which are parallel to the rafters, are marked by dots. The curved pieces C F, C′ F, are mere stays, to provide against casual irregularities of the load.

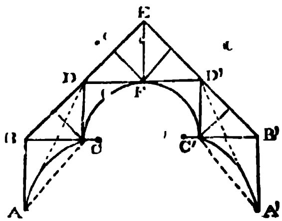


Fig. 211

The lines of resistance of the *primary truss* are the horizontal line D D′, and the dotted lines A D, A′ D′. The diagram of forces is formed thus:—In fig. 212, draw O H horizontal, H G vertical, and O G ‖ A D. Take H G to represent 3-8ths of the weight of the truss with its load; then will O H represent the horizontal stress, and O G the oblique thrust exerted along D A against the abutment.

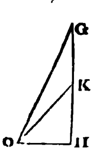


Fig. 212

The dotted line D A is the line of resistance of a frame or compound strut, consisting of the four struts A B, A C, B D, and C D, and the tie B C. The stresses on these pieces are represented as follows:—

the tension on B C, by O H (fig. 212.)
the thrusts along B D and A C, by O K ‖ B D ‖ A C; (K bisects H G);
the thrust on D C by K H = $\frac{3}{16}$ths of gross load;
the thrust on B A by $1\frac{2}{3}$ K H = $\frac{5}{16}$ths of gross load.

D E D′ forms a secondary truss, loaded at E with one-quarter of the gross load; D D′ is the tie of this truss as well as the straining piece of the primary truss; and the tension arising from the action of the secondary truss is to be subtracted from the thrust due to the action of the primary truss, to find the resultant thrust along D D′, which is thus found to be represented by $\frac{1}{3}$ O H. The thrust along E D is represented by $\frac{2}{3}$ O K.

240. Strength of Tie-Beams, Strut-Beams, and Bent Struts.—Let H be the greatest direct working stress, whether tension or thrust, along the line of resistance of a given piece whose breadth is b and depth h; M the greatest working bending moment, whether arising from a transverse load, or from the neutral axis of the piece not coinciding with the line of resistance (in which latter case M = H × greatest distance of the neutral axis from the line of resistance); f the greatest safe working intensity of stress; then,

$$f' = \frac{H}{b\,h} + \frac{6\,M}{b\,h^2}; \ldots\ldots\ldots\ldots\ldots\ldots (1.)$$

and if h has been fixed beforehand, b is given by the formula

$$b = \left(\frac{H}{h} + \frac{6\,M}{h^2}\right) \div f'. \ldots\ldots\ldots\ldots\ldots (2.)$$

As already stated, $f' = 1{,}000$ lbs. per square inch in ordinary carpentry.

341. **Bridge-Trusses.**—A bridge-truss is usually one of two or more parallel frames of carpentry, which act as girders, in supporting the cross-beams or joists of the platform of a bridge. (Article 336, p. 465.) The principal struts which it contains may spring either from a tie-beam, like the rafter of a roof, from iron sockets connected by means of a tie-rod, or from suitable piers and abutments of timber or stone. The most usual elementary figures of bridge-trusses are, like those of roof-trusses, the triangle (fig. 207), and the trapezoid (fig. 209); and the principles of their stability and equilibrium are the same, except that in a bridge-truss, special provision must be made for the unequal distribution of the load, both transversely and longitudinally. (See p. 806.)

I. *Load Unequal Transversely.*—This case occurs chiefly in bridges for double lines of railway, when one track is loaded and the other unloaded. The proportions in which the rolling load is distributed over the girders, when there are only two of them, is simply the inverse ratio of the horizontal distances of its centre of gravity from the two girders (Article 112, p. 171), but there are often more than two girders, most frequently four; and then, in order to determine the proportions in which the load is distributed over them, the assumption is made that the cross-beams remain sensibly straight; so that the difference between the deflections of any two of the girders, and consequently the difference between the shares of the load borne by them, is proportional simply to the distance between them.

To illustrate the application of this, let the girders, and the rolling load which by means of a cross-beam is made to rest on them, be arranged in cross-section as follows:—

W'

A B O C D

W' denotes the position of the centre of gravity of the rolling load; O the centre line of the platform; A, B, C, D, the four girders Then,

the *mean* share of the rolling load borne by each girder will be $W' \div 4$.

To find the deviations from that mean share, let

$$O\,B = z_1;\ O\,A = z_2;\ O\,C = -z_1;\ O\,D = -z_2;$$

and let the *horizontal* distance from O to W′ be z_0.

The deviation from the mean of the load on any girder whose distance from the centre line is z must be $a\,z$; a being a co-efficient to be determined by the condition that the moment of W′ relatively to O is equal and opposite to the sum of the moments of the resistances of the beams relatively to the same axis. This condition, expressed in symbols, gives $W' z_0 = 2\,a\,(z_1^2 + z_2^2)$; whence

$a = \frac{W' z_0}{2\,(z_1^2 + z_2^2)}$; and the shares of the rolling load on the four girders are as follows:—

$$\left.\begin{array}{ll}
\text{on A;} & \frac{W'}{4} + a\,z_2 = W'\left(\frac{1}{4} + \frac{z_0\,z_2}{2\,(z_1^2 + z_2^2)}\right); \\
\text{on B;} & \frac{W'}{4} + a\,z_1 = W'\left(\frac{1}{4} + \frac{z_0\,z_1}{2\,(z_1^2 + z_2^2)}\right); \\
\text{on C;} & \frac{W'}{4} - a\,z_1 = W'\left(\frac{1}{4} - \frac{z_0\,z_1}{2\,(z_1^2 + z_2^2)}\right); \\
\text{on D;} & \frac{W'}{4} - a\,z_2 = W'\left(\frac{1}{4} - \frac{z_0\,z_2}{2\,(z_1^2 + z_2^2)}\right).
\end{array}\right\} \quad \ldots(1.)$$

When the share of the load on D, as often happens, proves to be *negative*, it shows that the girder furthest from the loaded track is *pulled upwards* by the platform.

As a numerical example, let the bridge be one under an ordinary narrow gauge railway, and let the four girders be exactly under the four rails respectively; so that we may make, with sufficient accuracy for the present purpose,

$$z_1 = 3 \text{ feet};\ z_2 = 8 \text{ feet};\ z_0 = 5\tfrac{1}{2} \text{ feet};$$

then,

$$\begin{array}{rl}
\text{load on A} = & W'\left(\frac{1}{4} + \frac{22}{73}\right) = +\cdot 551\,W' \\
\text{,,}\quad B = & W'\left(\frac{1}{4} + \frac{33}{292}\right) = +\cdot 363\,W' \\
\text{,,}\quad C = & W'\left(\frac{1}{4} - \frac{33}{292}\right) = +\cdot 137\,W' \\
\text{,,}\quad D = & W'\left(\frac{1}{4} - \frac{22}{73}\right) = -\cdot 051\,W'
\end{array}$$

These results have been verified by careful experiments on a great scale.

The most important of them practically is the share of the load on A, being the greatest share. In order to arrange the girders so that this share shall not exceed *one-half*, the following equation should be fulfilled:—

$$z_1^2 = 2\, z_0\, z_2 - z_2^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.)$$

For example, let $z_0 = 5\frac{1}{2}$ feet; $z_2 = 10$ feet; then $z_1 = \sqrt{10}$ $= 3{\cdot}16$ feet.

II. *Load Unequal Longitudinally.*—This sort of inequality must be provided for in every case in which the figure of the truss has more sides than three.

The most important example in practice is that of the trapezoidal truss, whether springing from a tie-beam, as in fig. 209, p. 470, or from a pair of abutments, or from sockets connected by means of a tie-rod.

There are two means of enabling the truss to resist a partial load: by the stiffness of a longitudinal beam, and by diagonal bracing.

The longitudinal beam is either the tie-beam, or, in the absence of a tie-beam, a beam resting on the top of the truss, and bolted to the straining-piece F in the figure.

Let c denote the half-span of the truss; x, the distance of the points 4 and 1 from the middle of the truss.

Let a partial load W′ be applied at one of these points, the other being unloaded. Then the longitudinal beam has to resist a bending action, which is greatest at the loaded point and at the unloaded point, producing convexity downwards at the loaded point, and upwards at the unloaded point: the bending moment has the following value:—

$$M' = W'\, x\, (c - x) \ldots (3.)$$

and the stress produced by it must be taken into account in fixing the dimensions of the longitudinal beam. For example, if $x = c \div 3$, $M' = W'\, c \div 9$.

To provide resistance to a partial load by *diagonal bracing*, there should be two diagonal struts, in the positions shown by the dotted lines 4 5 and 6 1 in fig. 209; 4 5 to act when the partial load is on 4, and 6 1 when the partial load is on 1. The greatest thrust S along either of them is given by the following formula:—Let k be the depth of the truss, from the centre line of F to the centre line of B. Then

$$S = W' \frac{c - x}{2\, c\, k} \cdot \sqrt{4\, x^2 + k^2} \ldots\ldots\ldots\ldots (4.)$$

342. **Compound Bridge-Truss.** (*A. M.*, 160.)—The general nature of a compound truss has been explained in Article 121, p. 184. Fig. 213 is a skeleton diagram of a compound timber bridge-truss, on the principle of those of the celebrated bridge of Schaffhausen.

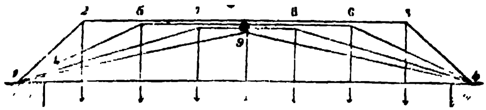


Fig. 213

It consists of four elementary trusses, viz.:—

1 2 3 4	loaded at	2	and	3,
1 5 6 4	"	5	"	6,
1 7 8 4	"	7	"	8,
1 9 4	"	9;		

but all those trusses have the same tie-beam, 1 4; and the pull along that tie-beam is the sum of the pulls due to the four trusses.

The vertical lines represent suspending-pieces, from which the tie-beam is hung. The tie-beam supports the cross-beams of the platform.

An arrangement of struts similar to that in the figure, but without the tie-beam or suspending-pieces, and supporting the platform above, is often used for timber bridges with abutments. Stay-pieces, however, are required, nearly in the position of the upper parts of the suspending-pieces in the figure, to give sufficient stiffness to the struts.

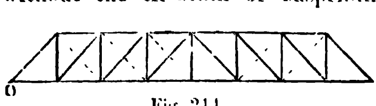


Fig. 214

343. **Diagonally-braced Girder.**—This sort of girder, of which fig. 214 is a skeleton diagram, was first introduced in America by Mr. Howe. The two horizontal bars, or "booms," resist the bending moment of the load; they are made of layers of planks set on edge, and bolted together so as to break joint, as in the built ribs of Article 333, p. 464. The shearing action of the load is resisted by the vertical suspending-pieces (which are iron rods), and the diagonal timber struts, which abut into iron sockets, as shown on a larger scale in fig. 215. In the latter figure A is the upper or compressed boom, and B the lower or extended boom; C

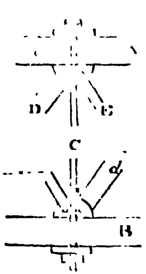


Fig. 215.

a suspending-rod; D, d, struts sloping up towards the middle of the span, and indicated by plain lines in fig. 214; E, e, struts sloping up towards the nearest point of support, and indicated by dotted lines in fig. 214.

The diagonals shown by full lines are all that would be required if the load were always uniformly distributed over the girder. Those shown by dots are necessary in order to resist travelling loads.

The actions of the load on this girder are computed by the method already explained in Article 160, pp. 240 to 243, as applied to a beam loaded at detached points. The formula for the bending moment at any cross-section has already been given in Article 161, Case VIII., p. 247. In computing the shearing force, regard must be had to the action of a travelling load, as explained in Article 161, Case IX., pp. 247, 248.

The following are the most convenient formulæ in practice. One of the points of support being numbered 0, the joints of the upper boom are to be numbered consecutively from that end of the girder towards the middle, as in fig. 214:—

Let n denote the number of any joint, and N the total number of divisions in the beam. (In the figure $N = 8$; and for the middle joint, $n = 4$. When N is odd, there is no middle joint.)

Let k denote the height of the girder, measured from centre to centre of the horizontal booms;

l, its span; so that $l \div N$ is the length of a division;

s, the length of a diagonal, measured along its line of resistance,

$$= \sqrt{k^2 + \frac{l^2}{N^2}};$$

w, the uniform steady load upon each joint;

w', the greatest travelling load upon each joint.

The divisions of the horizontal booms are to be numbered 1, 2, 3, 4, from the ends towards the middle; so that in fig. 214, Division No. 1 of the upper boom lies between 1 and 2; Division No. 1 of the lower boom lies between 0 and the suspending-rod 1, &c.

Suspending-rods and diagonals are designated by the number of the joint where their *upper* ends meet; thus, in fig. 215, if n be the number of the rod C, it is also the number of the larger diagonal D, and the smaller diagonal E; while the number of d is $n + 1$, and that of e, $n - 1$.

Let H_n be the thrust and tension along the division n of the upper and lower booms;

V_n, the tension on the vertical rod n;

T_n, the thrust on the large diagonal n;

t_n, the thrust on the small diagonal n.

Then

$$H_n = \frac{(w+w')l}{k} \cdot \frac{n(N-n)}{2N}; \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1.)$$

V_n (when the platform is *hung* from the girder, for all except the middle rod) $= w\left(\frac{N+1}{2} - n\right) + w' \cdot \frac{(N-n)(N-n+1)}{2N}; \ldots (2.)$

V (when the platform is hung from the girder, for the middle rod) $= w + \frac{w'}{4} \cdot \left(\frac{N}{2} + 1\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.)$

(When the platform rests on the top of the girder, subtract $w + w'$ from each of the above values of V.)

$$T_n = \frac{ws}{k}\left(\frac{N+1}{2} - n\right) + \frac{w's}{k} \cdot \frac{(N-n) \cdot (N-n+1)}{2N}; \ldots\ldots (4.)$$

$$t_n = -\frac{ws}{k}\left(\frac{N-1}{2} - n\right) + \frac{w's}{k} \cdot \frac{n \cdot (n+1)}{2N} \ldots\ldots\ldots\ldots (5.)$$

When the last formula (5) gives a null or negative result, it shows that the smaller diagonal in the division in question is unnecessary.

A common inclination for the diagonals is 45°; the corresponding value of $s \div k$ is 1·414, and that of $l \div k$ is N.*

The following is a numerical example:—

Span 80 feet, in eight equal divisions; that is, $l = 80$; $N = 8$.

$k = 10$ feet; $s = 14{\cdot}14$ feet.

$w = 5000$ lbs.; $w' = 10{,}000$ lbs.

Platform hung below girder.

n	H	V	T	t
	lbs.	lbs.	lbs.	lbs.
1	52,500	52,500	74,235	negative.
2	90,000	38,750	54,792	negative.
3	112,500	26,250	37,[illegible]28	7,070
4	120,000	17,500	21,210	

The last column shows that, in the example chosen, the dotted diagonals are required in the two middle divisions only.

The value of H for $n = 4$ applies to the lower boom alone, as the upper boom has only three divisions on each side of the middle.

314. **Lattice-work Girders** of timber were first introduced by Mr. Ithiel Towne. The lattice-work consists of planks inclined at 45° to the horizon pinned together with treenails.

* See page 493.

A lattice girder, even without horizontal booms (as in fig. 216), is capable of supporting a certain load, provided its ends are made fast to stable piers; and, under these circumstances, its moment of resistance at any cross-section is simply the sum of the moments of resistance of the planks intersected by that cross-section. But this mode of construction is unfavourable both to economy and to stiffness. When horizontal booms are bolted to the lattice-work at its upper and lower edges, they may be considered, without sensible error, as sustaining all the bending moment, like those in the example of the last article; while the lattice-work bears the shearing action of the load, distributed with approximate uniformity amongst the bars or planks.

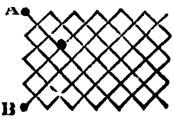


Fig. 216.

345. **Timber Arches.**—When a timber arch is exactly or nearly of the form of an equilibrated rib of uniform strength under the steady part of its load, and is subject besides to a rolling load, its strength is to be computed according to the methods of Article 180, pp. 296 to 314.

The usual form for timber arches is a segment of a circle; but the formulæ for a parabolic rib may be used in practice without material error. In almost every case the rib may be considered as *fixed in direction at the ends;* so that if the abutments are immoveable, the formulæ to be employed will be those of Problem IV., equations 30 to 38 B, pp. 305 to 308; and if the abutments are sensibly moveable, equation 40, p. 308, is to be used instead of equation 30.

In designing a timber arch, the greatest working deflection should be computed by the equation 61 of Article 180, p. 313; and the pieces of timber in the arch and superstructure should be proportioned as if the platform were to have an upward convexity or "camber," with a rise in the centre of the span equal to the calculated deflection. The result will be, that the platform will become horizontal, or nearly so, when fully loaded.

Semicircular timber ribs are now often employed to support roofs, for the sake of architectural appearance. In fig. 217, let A C B be a quadrant of such a rib, under a load uniformly distributed horizontally, O being its centre. Draw B D and A D tangents to the neutral layer at the springing and at the crown; bisect A D in E; then, if the arch be *jointed* or *hinged* at A and B, E B will be the direction of the thrust at B; and its horizontal component will be *half the load on the quadrant;* that is,

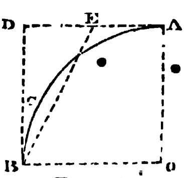


Fig 217.

$$H = \frac{w\,r}{2}; \qquad\qquad (1.)$$

r being the radius O A, and w the load per lineal unit of span. The greatest bending moment, on the same supposition, occurs at C, 30° above the springing. That moment tends to make *curvature sharper* at that point; and its value is

$$M = \frac{w\,r^2}{8}. \qquad\qquad (2.)$$

The value of the direct thrust is 2 H $= w\,r$, as given by equation **1.** By the use of these values in the formulæ of Article 340, p. 475, the proper scantling for the rib may be computed. The supposition of the rib being hinged at A and B is not perfectly realized in practice; but it will not lead to any error of importance.

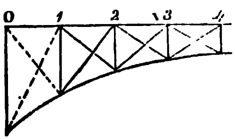


Fig. 218.

346. **Timber Spandrils.**—When timber arches support a level platform, each spandril in general contains a series of upright posts for transmitting the load from the platform to the arch. A horizontal beam on the top of each row of posts should have strength and stiffness sufficient to resist the load between each pair of posts.

To stiffen the frame transversely, the posts which stand side by side should have diagonal braces between them; the smallest transverse dimension of any brace not being less than about one-twentieth part of its length.

To stiffen the frame longitudinally, diagonal braces may be placed as in fig. 218. To find the stress which any one of those diagonal braces should be capable of resisting with safety, let the upright posts be numbered from one end of the arch to the middle, 0, 1, 2, 3, &c. (like the suspending-rods in Article 343). Let

N be the total number of longitudinal divisions in the platform.
n and $n + 1$, the numbers of the posts between which a given diagonal brace is situated.
s, its length, and k the difference of level of its ends.
w', the greatest *travelling* load on one post.
T, the greatest amount of thrust along the diagonal; then

$$T = \frac{w'\,s}{k} \cdot \frac{n\,(n+1)}{2\,N}. \qquad\qquad (1.)$$

For the diagonals between 0 and 1, indicated by dots in the figure, this expression is $= 0$; but nevertheless a pair of diagonals

may be placed there, of the same size with the smallest of those between 1 and 2, in order to give additional stiffness.

It is possible that when the arch is partially loaded with a travelling load, some of the upright pieces which, when the load is uniform, are posts, may have to act occasionally as suspending-pieces. To find whether this is the case for any given upright piece, let n be its number, and w'' the *dead* load resting upon it; then compute the value of the following expression:—

$$= w' \cdot \frac{n(n+1)}{2\,\mathrm{N}} - w''; \;\ldots\ldots\ldots\ldots\ldots\ldots (2.)$$

and if this is positive, it will give the greatest tension on the upright; if null or negative, it will show that the upright acts always as a strut or post, and never as a suspending-piece or tie.

Another mode of construction is to make all the diagonals iron tie-bolts. In this case equation 1 will give the greatest tension on any given bolt. The uprights will always act as posts, and the greatest load on each will be given by the following formulæ:—

$$\mathrm{V}' = w'' + w' \left\{ 1 + \frac{n(n-1)}{2\,\mathrm{N}} \right\}. \;\ldots\ldots\ldots\ldots (3.)$$

347. **Timber Bowstring Girder.** (Fig. 219.)—In a girder of this kind, a timber arch springs from a tie-beam, which supports the cross-beams of the platform, and is hung from the arch at intervals by vertical suspending-pieces or rods, with diagonal braces between them.

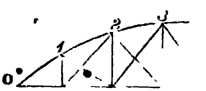


Fig. 219.

The tie-beam has to bear at once a tension equal to the horizontal thrust of the arch, and a bending action due to the load supported on it between a pair of suspending-pieces; and its strength depends on the principles explained in Article 340, p. 474.

The greatest tension on any suspending-piece is to be found by means of equation 3 of Article 346, above.

The greatest thrust along any diagonal is to be found by means of equation 1 of the same Article, p. 482.

The horizontal tie of a timber bowstring girder should never be made of iron, as its expansion and contraction would strain and at length destroy the timber arch.

348. **Timber Piers.**—A timber pier for supporting arches or girders may consist of any convenient number of posts, either vertical or slightly raking, and connected together by horizontal and diagonal braces.

Each post should be braced at every point where there is a joint in

it, and at additional points if necessary, in order that the distance between the braced points may not be greater than about 18 or 20 times the diameter of the post. (As to lengthening posts, see Article 315, p. 456.)

Should the pier have lateral thrust to bear, whether from the action of the wind or from that of the load upon the superstructure, the following principles are to be attended to:—

I. The posts at the base of the pier should, if possible, spread to such a distance from each other that the lateral thrust may cause *no tension* on any one of them. For example, conceive a pier of a timber viaduct to consist of two parallel rows of posts, let the greatest horizontal thrust in a direction perpendicular to the rows be H, acting at the height Y above the base of the pier, so that H Y is its moment; let W be the gross vertical load of the pier, and B the required distance from centre to centre between the two rows of posts at the base of the pier; then make

$$B = \frac{2 H Y}{W}; \qquad \text{(1.)}$$

and there will never be tension on any of the posts. If this arrangement be made, the *whole load* W will be concentrated on *one row of posts* when the greatest thrust acts. In other cases, the load on the row of posts furthest from the side of the pier on which the thrust acts will be,

$$\frac{W}{2} + \frac{H Y}{B}. \qquad \text{(2.)}$$

If the pier consists of more than two rows of posts, let n denote the number of rows, and let them be equidistant from each other, B being still the distance from centre to centre of the outside rows. Let P denote the share of the load which rests on the row of posts furthest from the side the thrust is applied to, and P′ the share which rests on the row nearest that side. Then

$$P = \frac{W}{n} + \frac{H Y (n-1)^2}{B \{(n-1)^2 + (n-3)^2 + \&c.\}}; \qquad \text{(3.)}$$

$$P' = \frac{W}{n} - \frac{H Y (n-1)^2}{B \{(n-1)^2 + (n-3)^2 + \&c.\}}; \qquad \text{(4.)}$$

the series in the denominator of the second term being carried on as long as the numbers in the brackets are positive.

The best value for B is found by making P′ = 0; that is to say,

$$B = \frac{H\,Y \cdot n\,(n-1)^2}{W\left\{(n-1)^2 + (n-3)^2 + \&c.\right\}}; \qquad .(5.)$$

in which cas

$$\frac{2\,W}{n} \qquad .(6.)$$

II. The horizontal and diagonal braces are to be calculated to resist the horizontal thrust, in the same manner that the suspending-pieces and diagonal struts of a diagonally braced girder are calculated to resist the shearing stress, supposing that shearing stress to be the same at all points of the girder, and = H.

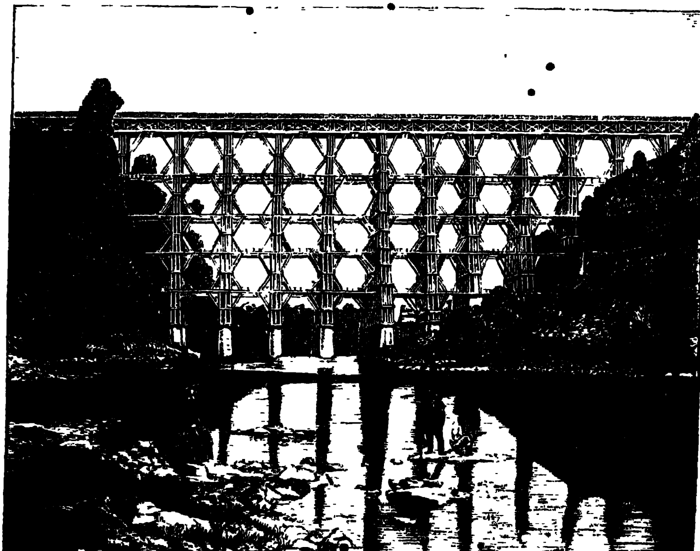


Fig. 220.—[Portage Bridge over the Genesee River, from a Photograph.]

349. **Centres for Arches.**—The use and general construction of centres for arches have already been explained in Article 279, p. 415. The present article relates to the figure and strength of the ribs or frames which support the laggings.

I. Action of Load on Centre.—The building of the arch should be carried up simultaneously at the two sides of the centre, so that the load on the centre may never be sensibly

unsymmetrical. The loading of the centre will thus advance from both ends towards the middle; and its most severe action, whether compressive, shearing, or bending, will take place just before the key-stones are driven into their places.

If there were no friction between the arch-stones, the load upon the centre could be computed exactly. The friction between them renders all formulæ for that purpose uncertain.

It is usually stated that the arch-stones do not begin to press against the centre until courses are laid the slope of whose beds is steeper than the angle of repose; that is to say, from 25° to 35°, or on an average, about 30°; but in order that this may be true, the lower part of the arch must be so thick as to have no tendency to *upset inwards*. A thickness equal to about one-tenth of the radius of curvature of the intrados is in general sufficient for that purpose; but still any accidental disturbance of the arch-stones may make them press against the centre.

Each successive course of arch-stones that is laid causes the pressure exerted by the previous courses against the centre to diminish; and when a semicircular arch is completed all but the key-stone, the stones whose beds slope less steeply than 30° have ceased to press against the centre, and that *even although there should be no friction.* In fact, when the load on the centre reaches its greatest amount, its action is nearly the same whether friction operates sensibly or not; and considering this fact, and also the fact that any errors in calculation caused by neglecting the friction of the stones on each other must be on the side of safety, it appears that for practical purposes it is sufficient to calculate the load on a centre as if the friction between the stones were insensible.

The following are the results:—

(1.) *General Case.*—Let w denote the weight *per lineal foot of the intrados* of the arch resting on a given rib of a centre.

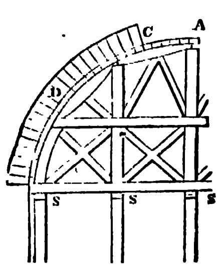


Fig. 221.

Let the co-ordinates of any point (such as D, fig. 221) in the intrados be measured from its highest point A; x being measured horizontally, and y vertically downwards.

Let x_0 and y_0 be the co-ordinates of the point C.

Let r be the radius of curvature of the intrados at the point D.

θ, its inclination to the horizon.

p, the *normal* pressure against the rib at the point D, *per lineal* foot of intrados; then, friction being insensible,

$$p = w \qquad \theta - \frac{1}{r}\int_{y_0}^{y} w\, d\, y; \qquad (1.)$$

and the greatest value of this is

$$w \cdot \cos \theta. \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1\text{ A.})$$

Let P be the total *vertical* load arising from the pressure of the arch-stones on the rib between C and D; then

$$P = \int_{x_0}^{x} p \, d\, x, \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.)$$

and the value of this, when the arch is complete all but the key-stone, is.

$$P_1 = \int_0^{x_1} p \, d\, x \text{ (making } y_0 = 0); \ldots\ldots\ldots\ldots(2\text{ A.})$$

x_1 being the horizontal distance from the middle of the span to a point for which $p = 0$.

If the rib, instead of resting on a series of posts, as in fig. 221, is supported *as a girder* on the abutments or piers of the arch, or on timber piers of its own,

Let c be the half-span of that girder;

M, the moment of flexure in the middle of the span; then

$$M = P\, c - \int_{x_0}^{x_1} p\, x\, d\, x; \ldots\ldots\ldots\ldots\ldots\ldots(3.)$$

and the greatest value of this is

$$M_1 = P_1\, c - \int_0^{x_1} p\, x\, d\, x \text{ (for } y_0 = 0). \ldots\ldots(3\text{ A.})$$

Formula 1 A. serves to compute the greatest load to be borne by the *laggings* or *bolsters;* equation 2 serves to compute the load on any vertical post, or the vertical component of the load on any given *back-piece,* or segment of the rib immediately under the laggings; and the total transverse load on such a piece is

$$P \sec \theta'. \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4.)$$

θ' being its inclination to the horizon.

Equation 2 A. gives the greatest vertical load on each half of the rib, and serves to compute the total strength required for its vertical supports; and equation 3 A serves to compute the strength required if the rib acts as a girder.

(2.) *Circular Arch not exceeding* 120°.—In an arch with a circular intrados, we have—

$$\begin{aligned} x &= r \sin \theta\ ; \ y = r\,(1 - \cos \theta\,); \\ x_0 &= r \sin \theta_0; \ y_0 = r\,(1 - \cos \theta_0); \\ x_1 &= r \sin \theta_1; \ y_1 = r\,(1 - \cos \theta_1); \end{aligned}$$

Let s, s_0, s_1, denote lengths of arcs measured from A in feet.

Let the weight per foot of intrados w be constant. Then the normal pressure per foot of intrados is

$$p = w\,(2 \cos \theta - \cos \theta_0) = w \cdot \frac{r - 2\,y + y_0}{r}; \;\ldots\ldots(5.)$$

and its greatest value for a given point,

$$w \cos \theta = w \cdot \frac{r - y}{r}. \;\ldots\ldots\ldots\ldots\ldots\ldots(5\text{ A.})$$

The vertical load between C and D is

$$\left.\begin{aligned} P &= w\,r\left\{\theta - \theta_0 - \sin \theta\,(\cos \theta_0 - \cos \theta)\right\} \\ &= w\left\{s - s_0 - \frac{x}{r}\,(y - y_0)\right\}; \end{aligned}\right\} \;\ldots(6.)$$

which, when the load is complete up to A, and D is *at the springing*, becomes

$$P_1 = w\,r\left\{\theta_1 - \sin \theta_1\,(1 - \cos \theta_1)\right\} = w\left\{s_1 - \frac{x_1\,y_1}{r}\right\}.\,(6\text{ A.})$$

in which last expression, the load on the half-rib is given in terms of its length, s_1, its *half-span*, x_1, and its *rise*, y_1.

The greatest moment of flexure, M_1, on a girder-rib of the half-span c, is as follows:—

$$\left.\begin{aligned} M_1 &= P_1\,c - w\,r^2\left(\frac{1}{6} + \frac{\cos^2 \theta_1}{2} - \frac{2 \cos^3 \theta_1}{3}\right) \\ &= w\left\{c\,s_1 - \frac{c\,x_1\,y_1}{r} - \frac{r^2}{6} - \frac{(r - y_1)^2}{2} + \frac{2(r - y_1)^3}{3r}\right\}. \end{aligned}\right\}(7.)$$

In employing these formulæ, it may often be convenient to use the following expressions for computing the radius r and length s of any given arc from its half-span x and rise y:—

$$r = \left(y + \frac{x^2}{y}\right) \div 2; \; s = x\left(1 + \frac{2}{3}\frac{y^2}{x^2} - \frac{y^4}{8\,x^4}\right) \text{ nearly.} \quad (8.)$$

The load on any arc of the rib may be represented graphically in the following manner:—

In fig. 222, let A B be a quadrant, described about O with a radius representing that of the intrados. Let C be the point up to which the arch has been built, and D any other point in the intrados.

Conceive that the *half* of the radius A O represents w, the weight per foot of intrados.

From C draw C E || A O; bisect C E in F, from which draw F H || O B; draw D G || A O; then will D G represent the normal pressure on each lineal foot of the rib at the point D; and the shaded area C D G F will represent the vertical component of the load on the rib between C and D, both in amount and in distribution; that is to say,

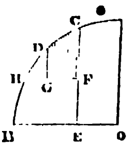


Fig ...

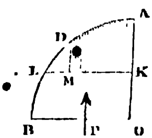


Fig. 223.

$$\tfrac{1}{2} \text{ A O} : w :: \text{D G} : p$$
$$:: \text{C D G F} : \text{P}.$$

The point H is that below which the arch stones cease to press on the rib, when the arch has been built up to the point C.

The case in which the rib is completely loaded, the arch being finished all but the key-stone, is represented by fig. 223. Bisect the vertical radius A O in K, and conceive A K to represent w; draw K L || O B; L will be a point below which the stones do not press on the rib (supposing the arch to extend so far); and at that point $\theta = 60°$. Let D be any point in the intrados; draw D M || A O; then

$$\text{A K} : w :: \text{D M} : p$$
$$:: \text{A D M K} : \text{P};$$

and if D is the *springing* of the arch, A D M K represents the vertical load on the half rib, P_1. If the arrow P in the figure represents the position of one of the two supports of a girder rib, O P $= c$ in equation 7.

(3.) *Circular Arch of* 120° *and upwards.*—Because the arch-stones below the point where the inclination of the intrados to the horizon is 60°, do not press upon the rib when the load is complete, the value of P_1 for $\theta_1 = \frac{1}{3}\pi$ applies also to all greater values of θ_1; it being understood that in every such case we are to make

$$x_1 = \sqrt{\frac{3}{4}} \cdot r = \cdot 866\, r;\ y_1 = \frac{r}{2};\ s_1 = 1 \cdot 0472\, r; \ldots (9.)$$

whatsoever the actual rise and span of the arch may be. This gives the following results:—

$$P_1 = \cdot 6142\, w\, r; \ldots\ldots\ldots\ldots\ldots\ldots\ldots (10.)$$

$$M_1 = w\, r \left(\cdot 6142\, c - \frac{5\, r}{24}\right). \ldots\ldots\ldots\ldots\ldots (11.)$$

(4.) *Non-circular Arch.*—Find the two points at which the intrados is inclined 60° to the horizon; conceive a circular arc drawn through them and through the crown of the intrados, and proceed as in Case 2, calculating r and s by the formulæ 8, from x and y, the co-ordinates of each of the two points where the inclination of the intrados is 60°. The results will be near enough to the truth for practical purposes.

II. STRIKING-PLATES AND WEDGES.—These terms are applied to the apparatus by which the centre is lowered after the arch has been completed. Fig. 224 represents a pair of striking-plates, A and B, with a compound wedge C between them. The lower striking-plate B is a strong beam, suitably notched on the upper side, and resting on the top of the pier or row of posts which forms one of the supports of a centre; the upper striking-plate A, notched on the under side, forms the base of part of the frame of the centre; and the wedge C keeps the striking-plates A and B asunder, being itself kept in its place by keys or smaller wedges driven behind its shoulders. When the centre is to be struck, those keys are driven out; and the wedge C being driven back with a mallet, allows the upper striking-plate to descend. In fig. 221, p. 486, S, S, S, represent the ends of pairs of striking-plates resting transversely on the posts or piles which support the entire centre. In some centres, of which examples will be given, the striking-plates lie longitudinally. In the centres introduced by Hartley, each lagging can be struck separately by lowering the wedges or screws which support it; so that striking-plates to support the entire centre are unnecessary. (See p. 493.)

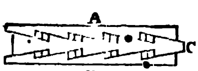


Fig. 224.

III. FRAMING OF CENTRES.—The back-pieces which form the upper edge of the rib, are usually supported at points from 10 to 15 feet asunder. In some examples, however, those points are as close as 5 or 6 feet.

It is essential that a centre should possess stiffness so great that polygonal frames of many sides and timber arches are unfit forms for its ribs, because of their flexibility. Such forms have been used, but have caused great difficulty and even danger in the construction of the arch. The kinds of framework which have been found to succeed are of three kinds, viz. :—

(1.) Direct supports from intermediate points; to be always employed when practicable.

(2.) Inclined struts in pairs;
(3.) Trussed girders;
} to be employed when intermediate points of support cannot be had close enough.

(1.) *Direct Supports* in a very simple form are illustrated by fig.

221, p. 486. Several rows of piles support a series of pairs of striking-plates. On the upper striking-plates rest the ribs, each of which consists of the following parts:—A *sill* or horizontal beam, a series of vertical posts directly over the piles, horizontal braces or *wales*, *diagonal braces* between the posts, *oblique struts* near the upper ends of the posts, to support intermediate points in the back-pieces, and the *back-pieces*. Besides giving stiffness to the posts, the diagonal braces answer the purpose of supporting a given part of the rib in case the pile vertically below it should give way.

Fig. 225 is a skeleton diagram of Hartley's centre for the Dee Bridge at Chester, in which the greater number of the supports consisted of struts, radiating in a fan-like arrangement from iron sockets or shoes on the tops of temporary stone piers, of which there were four in the total span of 200 feet. The struts were stiffened by means of wales at distances of from 10 to 12 feet apart vertically. The duty of back-pieces was done by two thicknesses of $4\frac{1}{2}$ inch planks. (See *Trans. Inst. Civ. Engs.*, vol. i.)

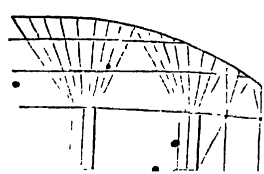


Fig. 225.

(2.) *Inclined Struts*, in pairs, are exemplified in fig. 226, which is a skeleton diagram of the centre of Waterloo Bridge. Each joint in the back-pieces, such as A, B, C, &c., was independently supported by a pair of struts of its own, springing from the striking-plates at F and F′. At each point where many of those struts

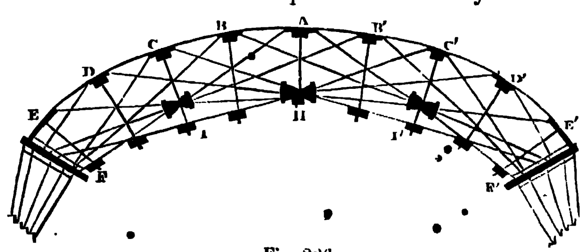


Fig. 226.

intersected each other, such as H, I, and I′, they were connected by abutting into one cast iron socket. At other points of intersection they were notched and bolted together. They were further stiffened by means of radiating pieces in pairs, whose positions are shown in the sketch. The striking-plates were longitudinal and inclined, and were supported on struts springing from the stepped [illegible] of the stone piers of the bridge.

(3.) *Trussed Girders*, as applied to centres, are illustrated (in fig. 227) by the centre of London Bridge. The sketch shows that for about one-fourth of the span at each side the support was direct,

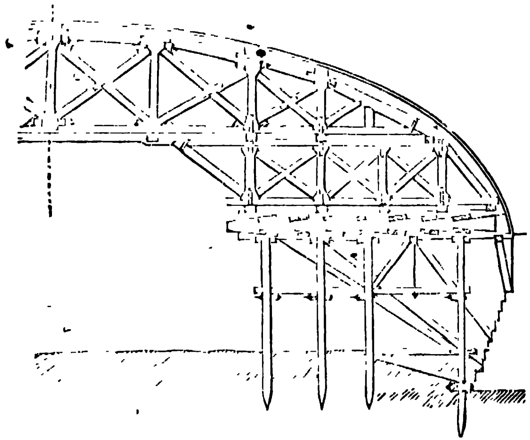


Fig. 227.

being given by vertical posts with diagonal braces between them; while across the middle half of the span the rib formed a diagonally-braced girder of great stiffness, its depth being about one-fourth of its span. The striking-plates were longitudinal and horizontal.

The ribs of a centre should be braced together transversely by horizontal and diagonal braces.

In framing centres it is desirable to use the pieces of timber in such a manner that they may be afterwards applied to other purposes.

(Addendum to Article 174, p. 280, and Article 312, pp. 450 to 453.)

349 A. **Resistance of Timber to Torsion.**—The following are the results of some recent experiments by M. Bouniceau on the resistance of timber to twisting and wrenching, extracted from a paper in the "*Annales des Ponts et Chaussées*" for 1861. The co-efficients are modified so as to suit the formulæ of M. de St. Venant for resistance to torsion, which are more correct than the ordinary formulæ employed in the original paper.

	Modulus of Rupture by Wrenching. f Lbs. on the Square Inch.	Modulus of Transverse Elasticity. C Lbs. on the Square Inch.
Red Pine of Prussia,	2,064	116,300
„ of Norway,	1,273	61,800
Elm,	1,863	76,000
Oak (of Normandy),	3,150	82,400
Ash,	1,956	76,000

For the formulæ of M. de St. Venant, and their investigation, see his notes to a recent edition of Navier's *Traité de la Resistance des Matériaux*.

The following are the formulæ applicable to square bars:—

Let h be the breadth and thickness of the bar.

M, the moment of torsion required to wrench it asunder; then

$$M = \cdot 208\, f\, h^3. \quad \text{..........................(1.)}$$

Also, let l be the length of the bar.

M′, any moment of torsion.

θ, the angle, stated in arc to radius unity, through which the bar is twisted by that moment; then

$$\theta = \frac{M'\, l}{\cdot 1405\, C\, h^4} \quad \text{..........................(2.)}$$

Addendum to Article 313, p. 480.

As to the most economical angles of inclination for diagonal braces, see papers by Mr. Bow in the *Civil Engineer and Architect's Journal* for 1861.

Addendum to Article 349, p. 490

349 B. **Striking of Centres by means of Sand.**—This process was first invented by M. Baudemoulin, perfected by M. de Sazilly, and carried into effect at the Bridge of Austerlitz, in Paris, by M. Bouziat.

The lower striking-plate consists of a timber platform, on which stand a number of vertical plate-iron cylinders, of nearly 1 foot in diameter, and 1 foot in height. The lower end of each cylinder fits on a circular wooden disc, about $\frac{3}{4}$ inch thick. About $1\frac{1}{4}$ inch above the base of each cylinder are four round holes, of about $\frac{3}{4}$ inch in diameter, stopped with corks. Each cylinder is filled about two-thirds or three-quarters full of clean dry sand; and upon the sand rests the lower end of a cylindrical wooden plunger loosely fitting the cylinder, which plunger is, in fact, the lower end of one of the upright posts of the framework of the centre. The joint between the plunger and the cylinder is stopped with plaster, to protect the sand from moisture. When the centre is to be struck, the corks are taken out of the cylinders, and the sand, running out of the holes, allows the centre to sink slowly and steadily. The sand, if necessary, may be loosened with a hook, to make it run freely; and it must be cleared away from the holes as it runs out.

(*Exposition Universelle*, 1862.—*Notices sur les Modèles, Cartes, et Dessins relatifs aux Travaux Publics.*)

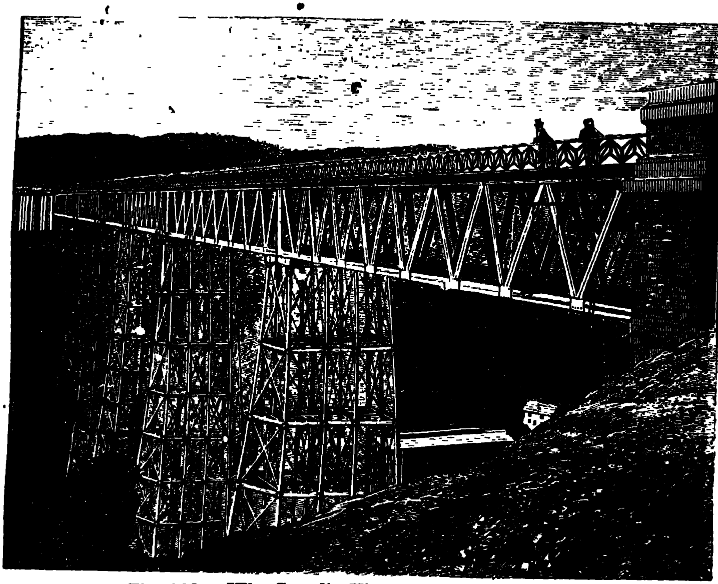


Fig. 228.—[The Crumlin Viaduct, from a Photograph.]

CHAPTER V.

OF METALLIC STRUCTURES.

SECTION I.—*Of Iron and Steel.*

350. **Sources and Classes of Iron in General.**—It would be foreign to the subject of the present treatise to enter into details as to the ores from which iron is obtained, and the processes of its manufacture. A brief summary, therefore, of those matters will alone be given, referring for more full information to such works as Percy *On Iron and Steel*; Lothian Bell's *Principles of the Manufacture of Iron and Steel*; Turner's *Metallurgy of Iron*; Ledebur & Wedding's Treatises; Phillips & Bauerman's *Manual of Metallurgy*; *The Journal of the Iron and Steel Institute*.

The Chemical equivalent of iron is 56 times that of hydrogen

The following are the most common conditions in which iron is found in its ores :—

		By Atoms.	By Weight.		Percentage of Iron.
I. *Native Iron*, being iron nearly pure, or combined with from one-fourth to one-hundredth part of its weight of nickel. This is very rare, and is found in detached masses, which are known, or supposed, to have fallen from the heavens,					80 to 100
II. Ferrous Oxide or Black Oxide of Iron,	Iron,	1	56	72	77·8
	Oxygen,	1	16		
Ferrous Oxide is only found in combination with other substances.					
III. Ferric Oxide or Red Oxide of Iron,	Iron,	2	112	160	70
	Oxygen,	3	48		
IV. Magnetic Oxide of Iron,	Iron,	3	168	232	72·4
	Oxygen	4	64		
V. Ferric Hydrate =					
Ferric Oxide, 2 atoms,	Iron,	4	224	374	60
	Oxygen,	6	144		
Water, 3 atoms,	Oxygen,	3			
	Hydrogen,	6	6		
VI. Carbonate of Iron =					
Ferrous Oxide, 1 atom,	Iron,	1	56	116	48·3
	Oxygen,	1	48		
Carbonic Anhydride, 1 atom,	Oxygen,	2			
	Carbon,	1	12		

Iron is found combined with sulphur, forming what is called *Iron Pyrites;* but that mineral is not available for the manufacture of iron; and it forms a pernicious ingredient in ores, or in the fuel used to smelt them, because of the weakening effect of sulphur upon iron. The same is the case with *Phosphate of Iron.*

The most abundant foreign ingredients found mixed with compounds of iron in its ores are siliceous sand and silicate of aluminium, or clay; next in abundance are the carbonates of calcium and magnesium. Amongst other foreign ingredients, which, though not abundant, have an influence on the quality of the iron produced, are carbon, manganese, arsenic, titanium, &c. Of these manganese and carbon alone are beneficial; for manganese gives increased strength to steel, and carbon assists in reducing the ore; all the rest are hurtful.

The most common *Ores of Iron* are the following:—

I. *Magnetic Iron Ore*, consisting of magnetic oxide of iron, pure, or almost pure, and containing 72 per cent. of iron, is found chiefly

in veins traversing the primary strata, and amongst plutonic rocks, and is the source of some of the finest qualities of iron, such as those of Sweden and the North-Eastern United States.

II. *Red Iron Ore* is ferric oxide, pure or mixed. When pure and crystalline, it is called *Specular Iron Ore*, or *Iron-glance;* when pure, or nearly so, and in kidney-shaped masses, showing a fibrous structure, it is called *Red Hæmatite;* when mixed with less or more clay and sand, it is called *Red Ironstone* and *Red Ochre.* It is found in various geological formations and is purest in the oldest. The purer kinds, iron-glance and hæmatite, produce excellent iron; for example, that of Nova Scotia.

III. *Brown Iron Ore* is ferric hydrate, pure or mixed. When compact and nearly pure, it is called *Brown Hæmatite;* when earthy and mixed with much clay, *Yellow Ochre.* It is found amongst various strata, especially those of later formations.

IV. *Carbonate of Iron*, when pure and crystalline, is called *Sparry* or *Spathose Iron Ore;* when mixed with clay and sand, *Clay Ironstone;* when clay ironstone is coloured black by carbonaceous matter, it is called *Black-band Ironstone.* These ores are found amongst various primary and secondary stratified rocks, and especially amongst those of the coal formation.

The proportion of earthy matter in the ordinary ores containing carbonate of iron ranges from 10 to 40 per cent.

The iron of Britain is manufactured partly from hæmatite, but chiefly from clay ironstone and black-band.

The extraction of iron from its ores consists of a combination of processes, which may be described in general terms as follows:—If the iron is in the state of carbonate, the carbonic acid is expelled by the agency of heat, leaving oxide of iron; the earthy constituents of the ore are removed by means of the chemical affinity of other earths (especially lime), forming a glassy refuse called *Slag;* the oxygen is taken away from the iron by means of the chemical affinity of carbon; and in certain processes, carbon, combined with the iron, is taken away by means of the chemical affinity of oxygen. There are also processes whose object is o combine the iron with certain proportions of carbon. The substances employed in the extraction of iron from its ore may be thus classed,—the *ore* itself; the *fuel*, which produces heat by its combustion, and supplies carbon; the *air*, which supplies oxygen for the combustion of the fuel; the *flux* (generally lime), which promotes the fusion of the ore, and combines with its earthy constituents.

Formerly the gases produced by the action in the blast furnace were burned at the top and thus were allowed to go to waste, now the furnaces are closed at the top and the waste gases utilised for heating the blast and raising steam, and are also

treated chemically whereby many bye products are obtained, such as ammonia, tar, &c.

The furnaces vary in height, some being about 60 feet high and 20 feet wide. The pressure of the blast about 5 lbs. to the square inch, and the temperature of the blast about 1,200° F.

The metallic products of the iron manufacture are of three kinds; *malleable* or *wrought iron*, being pure or nearly pure iron; *cast iron* and *steel*, being certain compounds of iron with carbon.*

351. **Impurities of Iron.**—The strength and other good qualities of these products depend mainly on the *absence of impurities*, and especially of certain substances which are known to cause brittleness and weakness, of which the most important are, sulphur, phosphorus, silicon, calcium, and magnesium.†

Sulphur and (according to Mushet) calcium, and probably also magnesium, make iron "*red short;*" that is, brittle at high temperatures; phosphorus and (according to Mushet) silicon make it "*cold short;*" that is, brittle at low temperatures. These are both serious defects; but the latter is the worse.

Sulphur comes in general from coal or coke used as fuel. Its pernicious effects can be avoided altogether by using fuel which contains no sulphur; and hence the strongest and toughest of all iron is that which is melted, reduced, and puddled either with charcoal, or with coal or coke that is free from sulphur. As to the artificial removal of sulphur, see the preceding Article.

Phosphorus comes in most cases from phosphate of iron in the ore, or from phosphate of lime in the ore, the fuel, or the flux. The ores which contain most phosphorus are those found in strata where animal remains abound, such as those of the oolitic formation.

Calcium and *Silicon* are derived respectively from the decomposition of lime and of silica by the chemical affinity of carbon for their oxygen. The only iron which is entirely free from these impurities is that which is made by the reduction of ores that contain neither silica, nor lime, such as pure magnetic iron ore, pure hæmatite, or pure sparry iron ore.

If either of those earths be present in the ore, the other must be added as a flux, to form a slag with it; and a small portion of each of them will be deoxidated, the bases uniting with the iron. This is a defect of earthy ores for which no remedy is yet known.

* According to some views recently set forth, *nitrogen* is one of the essential constituents of steel; but this wants confirmation.

† For a full discussion of the influence of impurities on iron, see *Introduction to the Study of Metallurgy*, by Professor Roberts-Austen.

The statements made relative to calcium are applicable also to magnesium.

The effect of aluminium upon iron is not known with certainty.

352. **Cast Iron** is the product of the process of *smelting* iron ores. In that process the ore in fragments, mixed with fuel and with flux, is subjected to an intense heat in a blast-furnace, and the products are *slag*, or glassy matter formed by the combination of the flux with the earthy ingredients of the ore, and *pig iron*, which is a compound of iron and carbon, either unmixed, or mixed with a small quantity of uncombined carbon in the state of plumbago.

The ore is often *roasted* or calcined before being smelted, in order to expel carbonic acid and water.

The proportions of ore, fuel, and flux are fixed by trial; and the success of the operation of smelting depends much on those proportions. The flux is generally limestone, from which the carbonic acid is expelled by the heat of the furnace; while the lime combines with the silica and alumina of the ore. If the ore contains carbonate of lime, less lime is required as a flux. If either lime or silica is present in excess, part of the earth which is in excess forms a glassy compound with oxide of iron, which runs off amongst the slag, so that part of the iron is wasted; and another part of that earth becomes reduced, its base combining with the iron and making it brittle, as has been stated in the preceding article; so that in order to produce at once the greatest quantity and best quality of iron from the ore, the earthy ingredients of the entire charge of the furnace must be in certain definite proportions, which are discovered for each kind of ore by careful experiment.

The total quantity of carbon in pig iron ranges from 2 to 5 per cent. of its weight.

Different kinds of pig iron are produced from the same ore in the same furnace under different circumstances as to temperature and quantity of fuel. A high temperature and a large quantity of fuel produce *grey cast iron*, which is further distinguished into No. 1, No. 2, No. 3, and so on; No. 1 being that produced at the highest temperature. A low temperature and a deficiency of fuel produce *white cast iron*. Grey cast iron is of different shades of bluish-grey in colour, granular in texture, softer and more easily fusible than white cast iron. White cast iron is silvery white, either granular or crystalline, comparatively difficult to melt, brittle, and excessively hard.

It appears that the differences between those kinds of iron depend not so much on the total quantities of carbon which they

contain as on the proportions of that carbon which are respectively in the conditions of mechanical mixture and of chemical combination with the iron. Thus, grey cast iron contains *one* per cent., and sometimes less, of carbon in chemical combination with the iron, and from *one* to *three* or *four* per cent. of carbon in the state of plumbago in mechanical mixture; while white cast iron is a homogeneous chemical compound of iron with from 2 to 4 per cent. of carbon. Of the different kinds of grey cast iron, No. 1 contains the greatest proportion of plumbago, No. 2 the next, and so on.

There are two kinds of white cast iron, the *granular* and the *crystalline*. The granular kind can be converted into grey cast iron by fusion and slow cooling; and grey cast iron can be converted into granular white cast iron by fusion and sudden cooling. This takes place most readily in the best iron. Crystalline white cast iron is harder and more brittle than granular, and is not capable of conversion into grey cast iron by fusion and slow cooling. It is said to contain more carbon than granular white cast iron; but the exact difference in their chemical composition is not yet known.

Grey cast iron, No. 1, is the most easily fusible, and produces the finest and most accurate castings; but it is deficient in hardness and strength; and, therefore, although it is the best for castings of moderate size, in which accuracy is of more importance than strength, it is inferior to the harder and stronger kinds, No. 2 and No. 3, for large structures.

353. **Strength of Cast Iron.**—Something has been already stated as to the comparative strength of different kinds of cast iron. It may be laid down as a general principle, that the presence of plumbago renders iron comparatively weak and pliable, so that the order of strength among different kinds of cast iron from the same ore and fuel is as follows:—

Granular white cast iron.
Grey cast iron, No. 3.
" " No. 2.
" " No. 1.

Crystalline white cast iron is not introduced into this classification, because its extreme brittleness makes it unfit for use in engineering structures.

Granular white cast iron also, although stronger and harder than grey cast iron, is too brittle to be a safe material for the entire mass of any girder, or other large piece of a structure; but it is used to form a hard and impenetrable *skin* to a piece of grey cast iron by the process called *chilling*. This consists in lining the

portion of the mould where a hardened surface is required with suitably shaped pieces of iron. The melted metal, on being run in, is cooled and solidified suddenly where it touches the cold iron; and for a certain depth from the chilled surface, varying from about $\frac{1}{8}$th to $\frac{1}{2}$ inch in different kinds of iron, it takes the white granular condition, while the remainder of the casting takes the grey condition.

Even in castings which are not chilled by an iron lining to the mould, the outermost layer, being cooled more rapidly than the interior, approaches more nearly to the white condition, and forms a *skin* harder and stronger than the rest of the casting.

The best kinds of cast iron for large structures are No. 2 and No. 3; because, being stronger than No. 1, and softer and more flexible than white cast iron, they combine strength and pliability in the manner which is best suited for safely bearing loads that are in motion.

As to the comparative strength of irons melted by the cold blast and by the hot blast, it appears from the experiments of both Fairbairn and Hodgkinson, that with the same kind of ore and fuel, No. 1 cold blast is in general superior to No. 1 hot blast iron; No. 2 hot and cold blast are about equally good; No. 3 hot blast is in general superior to No. 3 cold blast; and the average quality of the iron on the whole is nearly the same with the hot as with the cold blast.

A strong kind of cast iron called *toughened cast iron*, is produced by the process, invented by Mr. Morries Stirling, of adding to the cast iron, and melting amongst it, from one-fourth to one-seventh of its weight of wrought iron scrap.

The manner in which the strength of cast iron depends on the absence of impurities from the ore and fuel has already been mentioned in Article 351, p. 497.

Various mixtures of different qualities of iron have been recommended by different engineers as materials for large castings. (On this point see the *Report on the Application of Iron to Railway Structures*, p. 265.) For example, Fairbairn recommended the following combination:—

Lowmoor, No. 3, ...	30	per cent.
Blaina, or Yorkshire, No. 2, ...	25	„
Shropshire, or Derbyshire, No. 3, ...	25	„
Good old malleable scrap, ...	20	„
	100	„

Sir Charles Fox recommended a combination of two-thirds Welsh cold blast iron, and one-third Scotch hot blast iron, the

latter being manufactured from equal proportions of black-band and hæmatite ores. But both these and other engineers agreed in considering that the best course for an engineer to take in order to obtain iron of a certain strength for a proposed structure was, not to specify to the founder any particular mixture, but to specify a certain minimum strength which the iron should exert when tested by experiment.

The strength of cast iron to resist cross breaking was found by Fairbairn to be increased by *repeated meltings* up to the *twelfth*, when it was greater than at the first in the ratio of 7 to 5 nearly. After the twelfth melting that sort of strength rapidly fell off.

The resistance to crushing went on increasing after each successive melting; and after the *eighteenth* melting it was double of its original amount, the iron becoming silvery white and intensely hard.

The transverse strength of No. 3 cast iron was also found by Fairbairn not to be diminished by raising its temperature to 600° Fahr. (being about the temperature of melting lead). At a red heat its strength fell to two-thirds.

The strength of cast iron of every kind is marked by two properties; the smallness of the tenacity as compared with the resistance to crushing, and the different values of the modulus of rupture of the same kind of iron in bars torn directly asunder, and in beams of different forms when broken across. These circumstances have already been referred to in Article 157, p. 235, Article 164, pp. 256 to 258, and Article 166, p. 261. The variations in the modulus of rupture for beams of different figures arise in all probability from the greater tenacity of the skin, as compared with the interior of the casting; for an experiment on a bar torn directly asunder shows the least tenacity of its internal particles; while experiments on beams broken across show the tenacity of some layer which is nearer to or further from the skin according to the form of cross-section.

Intense cold makes cast iron brittle; and sudden changes of temperature sometimes cause large pieces of it to split.

The *proof strength* of cast iron has been shown to be about *one-third of the breaking load*, by experiments already mentioned in the note to p. 221. The usual *factor of safety* for the working load on railway structures of cast iron is *six*. (See Article 143, p. 222.)

In addition to the data in the tables at the end of the volume, the following table gives results as to the strength of cast iron, extracted and condensed from the experiments by the same authorities. All the coefficients are in lbs. on the square inch.

Kinds of Iron.		Direct Tenacity.	Resistance to Direct Crushing.	Modulus of Rupture of Square Bars.	Modulus of Elasticity.
No. 1. Cold blast, ...	from	12,694	56,455	36,693	14,000,000
	to	17,466	80,561	39,771	15,380,000
No. 1. Hot blast, ...	from	13,434	72,193	29,889	11,539,000
	to	16,125	88,741	35,316	15,510,000
No. 2. Cold blast, ...	from	13,348	68,532	33,453	12,586,000
	to	18,855	102,408	39,609	17,036,000
No. 2. Hot blast, ...	from	13,505	82,734	28,917	12,259,000
	to	17,807	102,030	38,394	16,301,000
No. 3. Cold blast, ...	from	14,200	76,960	35,881	14,281,000
	to	15,508	115,400	47,061	22,908,000
No. 3. Hot blast, ...	from	15,278	101,831	35,640	15,852,000
	to	23,468	104,881	43,497	22,733,000
No. 4. Smelted by coke without sulphur,		—	--	41,715	—
Toughened cast iron, ...	from	23,461	129,876	—	—
	to	25,764	119,457	—	—
No. 3. Hot blast after first melting,......		—	98,560	39,690	—
" " " twelfth melting,...		—	163,744	56,060	—
" " " eighteenth " ...		—	197,120	25,350	—
(Malleable Cast Iron.—See p. 587.)					

It is to be understood that the numbers in one line of the preceding table do not necessarily belong to the *same specimen* of iron, each number being an *extreme* result for the kind of iron specified in the first column.

The *modulus of rupture of cast iron by wrenching*, according to an average of several experiments, is

27,700 lbs. on the square inch.

If this be denoted by f', the *wrenching moments* of cast iron shafts are

for *round bars* of the diameter h, $\cdot 196\, f'\, h^3$
for *square bars* of the dimensions $h \times h$, $\cdot 208\, f'\, h^3$ } in inch-lbs.

for *hollow cylinders*; diameter, outside, h_1, inside, h_0, $\cdot 196 f' \cdot \dfrac{h_1^4 - h_0^4}{h_1}$

(See also p. 605.)

354. **Castings for Works of Engineering.**—The quality of the iron suitable for such castings has already been discussed.

As to appearance, it should show on the outer surface a smooth, clear, and continuous skin, with regular faces and sharp angles. When broken, the surface of fracture should be of a light bluish-grey colour and close-grained texture, with considerable metallic lustre; both colour and texture should be uniform, except that near the skin the colour may be somewhat lighter and the grain closer; if the fractured surface is *mottled*, either with patches of darker or lighter iron, or with crystalline patches, the casting will be unsafe;

and it will be still more unsafe if it contains air-bubbles. The iron should be soft enough to be slightly indented by a blow of a hammer on an edge of the casting.

Castings are tested for air-bubbles by ringing them with a hammer all over the surface.

Cast iron, like many other substances, when at or near the temperature of fusion, is a little more bulky for the same weight in the solid than in the liquid state, as is shown by the solid iron floating on the melted iron. This causes the iron as it solidifies to fill all parts of the mould completely, and to take a sharp and accurate figure.

The solid iron contracts in cooling from the melting point down to the temperature of the atmosphere, by $\frac{1}{96}$th part in each of its linear dimensions, or *one-eighth of an inch in a foot;* and therefore patterns for castings are made larger in that proportion than the intended pieces of cast iron which they represent.

In designing patterns for castings, care must be taken to avoid all abrupt variations in the thickness of metal, lest parts of the casting near each other should be caused to cool and contract with unequal rapidity, and so to split asunder or overstrain the iron.

Iron becomes more compact and sound by being cast under pressure; and hence cast iron cannon, pipes, columns, and the like, are stronger when cast in a vertical than in a horizontal position, and stronger still when provided with a *head*, or additional column of iron, whose weight serves to compress the mass of iron in the mould below it. The air bubbles ascend and collect in the head, which is broken off when the casting is cool.

Care should be taken not to cut or remove the skin of a piece of cast iron at those points where the stress is intense.

Cast iron expands in linear dimensions by about 1-900th, or ·00111, in rising from the freezing to the boiling point of water; being at the rate of ·00000617 for each degree of Fahrenheit's scale, or about ·0004 for the range of temperature which is usual in the British climate. Every structure containing cast iron must be so designed that the greatest expansion and contraction of the castings by change of temperature shall not injure the structure. (See p. 804.)

355. **Wrought or Malleable Iron** in its perfect condition is simply pure iron. It falls short of that perfect condition to a greater or less extent owing to the presence of impurities, of which the most common and injurious have been mentioned, and their effects stated, in Article 351, p. 497; and its strength is in general greater or less according to the greater or less purity of the ore and fuel employed in its manufacture.

Malleable iron may be made either by direct reduction of the ore, or by the abstraction of the carbon and various impurities from

cast iron. The process of direct reduction is applicable to rich and pure ores only; and it leaves a slag or "cinder" which contains a large proportion of oxide of iron, and yields pig iron by smelting. The most economical and generally applicable process is that of removing the foreign constituents from pig iron; and for that purpose white pig iron (called "forge pig") is usually employed, partly because it retains less carbon on the whole than grey pig iron, and partly because it is unfit for making castings. The details of the process are very much varied; but the most important principle of its operation always is to bring the pig iron in a melted state into close contact with a quantity of air sufficient to oxidate all the carbon and silicon. The carbon escapes in carbonic oxide or carbonic acid gas; the silica produced by the oxidation of the silicon combines partly with protoxide of iron and partly with lime (which is sometimes introduced as a flux for it), and forms slag or "cinder." Chloride of sodium (common salt) is used to remove sulphur and phosphorus. In one form of the process this is accomplished by injecting jets of steam amongst the molten iron; the oxygen of the steam assists in oxidating the carbon and silicon, and the hydrogen combines with the sulphur and phosphorus. The surest method, however, of obtaining iron free from the weakening effects of sulphur and phosphorus is to employ ores and fuel that do not contain those constituents.

The most common form of the process of making malleable iron is *puddling*, in which the pig iron is melted in a reverberatory furnace, and is brought into close contact with the air by stirring it with a rake or "rabble." Some iron makers precede the process of puddling by that of "refining," in which the pig iron, in a melted state, has a blast of air blown over its surface. This removes part of the carbon, and leaves a white crystalline compound of iron and carbon called "refiners' metal." Others omit the refining, and at once puddle the pig iron; this is called "*pig boiling*." The removal of the carbon is indicated by the thickening of the mass of iron, malleable iron requiring a higher temperature for its fusion than cast iron. It is formed into a lump called a "loup" or "bloom," taken out of the furnace, and placed under a tilt hammer or in a suitable squeezing machine, to be "*shingled;*" that is, to have the cinder forced out, and the particles of iron welded together by blows or pressure.

The bloom is then passed between rollers, and rolled into a bar; the bar is cut into short lengths, which are fagotted together, reheated, and rolled again into one bar; and this process is repeated till the iron has become sufficiently compact and has acquired a fibrous structure.

In the Bessemer process, the molten pig iron, having been run

into a suitable vessel, has jets of air blown through it by a blowing machine. The oxygen of the air combines with the silicon and carbon of the pig iron, and in so doing produces enough of heat to keep the iron in a melted state till it is brought to the malleable condition; it is then run into large ingots, which are hammered and rolled in the usual way. This process has been most successful when applied to pig iron that is free from sulphur and phosphorus, such as that of Sweden and Nova Scotia.

Strength and toughness in bar iron are indicated by a fine, close, and uniform fibrous structure, free from all appearance of crystallization, with a clean bluish-grey colour and silky lustre on a torn surface where the fibres are shown.

Plate iron of the best kind consists of alternate layers of fibres crossing each other, and ought to be nearly of the same tenacity in all directions.

Malleable iron is distinguished by the property of *welding:* two pieces, if raised nearly to a white heat and pressed or hammered firmly together, adhering so as to form one piece. In all operations of rolling or forging iron of which welding forms a part, it is essential that the surfaces to be welded should be brought into close contact, and should be perfectly clean and free from oxide of iron, cinder, and all foreign matter.

In all cases in which several bars are to be fagotted or rolled into one attention should be paid to the manner in which they are "*piled*" or built together, so that the pressure exerted by the hammer or the rollers may be transmitted through the whole mass. If this be neglected, the finished bar or other piece may show flaws marking the divisions between the bars of the pile (as is often exemplified in rails).

Wrought iron, although it is at first made more compact and strong by *reheating* and hammering, or otherwise working it, soon reaches a state of maximum strength, after which all reheating and working rapidly makes it weaker (as will afterwards be shown by examples). Good bar iron has in general attained its maximum strength; and therefore, in all operations of forging it, whether on a great or small scale, by the steam-hammer or by that in the hand of the blacksmith, the desired size and figure ought to be given with the least possible amount of reheating and working.

It is still a matter of dispute to what extent and under what circumstances wrought iron loses its fibrous structure and toughness, and becomes *crystalline* and brittle. By some authorities it is asserted that all shocks and vibrations tend to produce that change; others maintain that only *sharp* shocks and vibrations do so; and others, that no such change takes place; but that the same piece of iron which shows a fibrous fracture, if gradually broken by

a steady load, will show a crystalline fracture, if suddenly broken by a sharp blow. The author of this work at one time made a collection of several journals of railway carriage axles, which, after running for two or three years, had broken spontaneously by the gradual creeping inwards of an invisible crack at the shoulder. The fracture of every one of these was wholly or almost wholly fibrous; while other axles from the same works, when broken by the hammer, showed some a fibrous and others a crystalline fracture.* It is certain, at all events, that iron ought to be as little as possible exposed to sharp blows and rattling vibrations.

It is of great importance to the strength of all pieces of forged iron that the *continuity of the fibres* near the surface should be as little interrupted as possible; in other words, that the fibres near the surface should lie in layers parallel to the surface. This principle is illustrated by the results of some experiments made by the author of this work on the fracture of axles. Two cylindrical wrought iron railway carriage axles, one rolled, the other fagotted with the hammer, of four inches in diameter, were taken; a pair of journals of two inches in diameter were formed on the ends of each axle, one journal being reduced to the smaller diameter entirely by turning, so that the fibres at the shoulder did not follow the surface, and the other as far as possible by forging with the hammer, only one-sixteenth of an inch being turned off in the lathe to make it smooth, so that the fibres at the shoulder followed the surface almost exactly. All the journals were then broken off by blows with a 16 lb. hammer; when those whose diameters had been reduced by turning broke off with the *first* blow; and of those which had been drawn down by forging, that of the rolled axle broke off with the *fifth* blow, that of the hammered axle with the *eighth*. (*Proceedings of the Inst. of Civil Engineers*, 1843.)

Another important principle in designing pieces of forged iron which are to sustain shocks and vibrations, is to avoid as much as possible abrupt variations of dimensions and angular figures, especially those with re-entering angles; for at the points where such abrupt variations and angles occur fractures are apt to commence. If two parts of a shaft, for example, or of a beam exposed to shocks and vibrations, are to be of different thicknesses, they should be connected by means of curved surfaces, so that the change of thickness may take place gradually, and without re-entering angles.

356. **Steel and Steely Iron.**—Steel, the hardest of the metals and the strongest of known substances, is a compound of iron with from

* Full sized drawings of the fractured surfaces of several of these axles are in the possession of the Institution of Civil Engineers.

0·5 to 1·5 per cent. of its weight of carbon. These, according to most authorities, are the only essential constituents of steel. (See Article 350, p. 497.)

The term "steely iron," or "semi-steel," may be applied to compounds of iron with less than 0·5 per cent. of carbon. They are intermediate in hardness and other properties between steel and malleable iron.

In general, such compounds are the harder and the stronger, and also the more easily fusible, the more carbon they contain; those kinds which contain less carbon, though weaker are more easily welded and forged, and from their greater pliability are the fitter for structures that are exposed to shocks.

Impurities of different kinds affect steel injuriously in the same way with iron. (See Article 351, p. 497.)

There are certain foreign substances which have a beneficial effect on steel. One 2,000th part of its weight of silicon causes steel to cool and solidify without bubbling or agitation; but a larger proportion is not to be used, as it would make the steel brittle. The presence of manganese in the iron, or its introduction into the crucible or vessel in which steel is made, improves the steel by increasing its toughness and making it easier to weld and forge; but whether the manganese remains in combination with the iron and carbon in the steel, or whether it produces its effects by its temporary presence only, is not known with certainty.

Steel is distinguished by the property of *tempering;* that is to say, it can be hardened by sudden cooling from a high temperature, and softened by gradual cooling; and its degree of hardness or softness can be regulated with precision by suitably fixing that temperature. The ordinary practice is, to bring all articles of steel to a high degree of hardness by sudden cooling, and then to soften them more or less by raising them to a temperature which is the higher the softer the articles are to be made, and letting them cool very gradually. The elevation of temperature previous to the "annealing" or gradual cooling is produced by plunging the articles into a bath of a fusible metallic alloy. The temperature of the bath ranges from 430° to 560° Fahr.

It is supposed that hard steel is analogous to granular white cast iron, being a homogeneous chemical compound of iron and carbon; that soft steel is analogous to grey cast iron, and is a mixture of a carburet of iron containing less carbon than hard steel with another carburet containing more carbon; and that slow cooling favours the separation of those two carburets.

Steel is made by various processes, which have of late become very numerous. They may all be classed under two heads, viz., adding carbon to malleable iron, and abstracting carbon from cast

iron. The former class of processes, though the more complex, laborious, and expensive, is preferred for making steel for cutting tools and other fine purposes, because of its being easier to obtain malleable iron than cast iron in a high state of purity. The latter class of processes is the best adapted for making great masses of steel and steely iron rapidly and at moderate expense. The following are some of the processes employed in making different kinds of steel:—

I. *Blister Steel* is made by a process called "*cementation*," which consists in imbedding bars of the purest wrought iron (such as that manufactured by charcoal from magnetic iron ore) in a layer of charcoal, and subjecting them for several days to a high temperature. Each bar absorbs carbon, and its surface becomes converted into steel, while the interior is in a condition intermediate between steel and iron. Cementation may also be performed by exposing the surface of the iron to a current of carburetted hydrogen gas at a high temperature. Cementation is sometimes applied to the surfaces of articles of malleable iron in order to give them a skin or coating of steel, and is called "*case-hardening*."

II. *Shear Steel* is made by breaking bars of blister steel into lengths, making them into bundles or fagots, and rolling them out at a welding heat, and repeating the process until a near approach to uniformity of composition and texture has been obtained. It is used for various tools and cutting implements.

III. *Cast Steel* is made by melting bars of blister steel in a crucible, along with a small additional quantity of carbon (usually in the form of coal tar) and some manganese. It is the purest, most uniform, and strongest steel, and is used for the finest cutting implements.

Another process for making cast steel, but one requiring a higher temperature than the preceding, is to melt bars of the purest malleable iron with manganese and with the whole quantity of carbon required in order to form steel. The quality of the steel as to hardness is regulated by the proportion of carbon. A sort of semi-steel, or steely iron, made by this process, and containing a small proportion of carbon only, is known as *homogeneous metal*.

IV. *Steel made by the air blast* is produced from molten pig iron by the Bessemer process (Article 355, p. 504) in two ways; either the blowing of jets of air through the iron is stopped at an instant determined by experience, when it is known that a quantity of carbon still remains in the iron sufficient to make steel of the kind required, or else the blast is continued until the carbon is all removed, so that the vessel is full of pure malleable iron in the melted state, and carbon is added in the proper proportion, along with manganese and silicon. The steel thus produced is run

into large ingots, which are hammered and rolled like blooms of wrought iron.

V. *Puddled Steel* is made by puddling pig iron (Article 355, p. 504), and stopping the process at the instant when the proper quantity of carbon remains. The bloom is shingled and rolled like bar iron.

VI. *Granulated Steel* (the invention of Captain Uchatius) is made by running melted pig iron into a cistern of water, over a wheel, which dashes it about so that it is found at the bottom of the cistern in the form of grains or lumps of the size of a hazel nut, or thereabouts. These are imbedded in pulverized hæmatite, or sparry iron ore, and exposed to a heat sufficient to cause part of the oxygen of the ore to combine with and extract the carbon from the superficial layer of each of the lumps of iron, each of which is reduced to the condition of malleable iron at the surface, while its heart continues in the state of cast iron. A small additional quantity of malleable iron is produced by the reduction of the ore. These ingredients being melted together, produce steel.

There are other processes for making steel in large quantities, for which see Appendix.

357. **Strength of Wrought Iron and Steel.**—Wrought iron, like fibrous substances in general, is more tenacious along than across the fibres; and its tenacity is greater than its resistance to crushing. The effect of the latter difference on the best forms of cross-section for beams has already been considered in Article 164, pp. 256 to 259, and will be further illustrated in the sequel.

The ductility of wrought iron often causes it to yield by degrees to a load, so that it is difficult to determine its strength with precision.

Wrought iron has its longitudinal tenacity considerably increased by rolling and wire-drawing; so that the smaller sizes of bars are on the whole more tenacious than the larger; and iron wire is more tenacious still, as the figures in the table of tenacity at the end of the volume show.

Wrought iron is weakened by too frequent reheating and forging; so that even in the best of large forgings, the tenacity is only about *three-fourths* of that of the bars from which the forgings were made, and sometimes even less.

As to the *effect of heat on the strength of wrought iron*, it has been shown by Fairbairn (*Useful Information for Engineers*, second series):—

I. That the tenacity of ordinary *boiler plate* is not appreciably diminished at a temperature of 395° Fahr., but that at a dull red heat it is diminished to about three-fourths.

II. That the tenacity of good *rivet iron* increases with elevation of temperature up to about 320° Fahr., at which point it is about one-third greater than at ordinary atmospheric temperatures; and that it then diminishes, and at a red heat is reduced to little more than one-half of its value at ordinary atmospheric temperatures.

The resistance of iron rivets to shearing is nearly the same with the tenacity of the best boiler plates.

As to the strength of wrought iron to resist crushing, see Article 157, p. 237.

Numerous experiments have been made on the tenacity of steel; but its other kinds of strength have been very little investigated. Its tenacity, like that of bar iron, is increased by rolling and wire-drawing.

The experiments already quoted in the note to Article 142, p. 221, have shown that the *proof strength* of wrought iron is almost exactly *one-third* of the breaking load.

The tables at the end of the volume give only average or extreme results as to the strength of wrought iron and steel; and therefore the following tables are here annexed, in which more details are given, but still in a very condensed form, the authorities being as denoted by the capital letters (see p. 513). (See also Addenda, p. 790.)

TABLE OF THE TENACITY OF WROUGHT IRON AND STEEL.

Description of Material.	Tenacity in lbs. per Square Inch. Lengthwise.		Crosswise.	Ultimate Extension.
MALLEABLE IRON.				
Wire—Very strong, charcoal, ...	114,000	Mo.		
Wire—Average, ...	86,000	T.		
Wire—Weak, ...	71,000	Mo.		
Yorkshire (Lowmoor),...	64,200	F.	52,490 F.	
,, from	66,390	N.		0·20
to	60,075	N.		0·26
Yorkshire (Lowmoor) and Staffordshire rivet iron, ...	59,740	F.	"	0·2 to 0·25
Charcoal bar, ...	63,620	F.		0·2
Staffordshire bar,... from	62,231	N.		·302
to	56,715	N.		·186
Yorkshire bridge iron,...	49,930	F.	43,940 F.	·04; ·029
Staffordshire bridge iron,	47,600	F.	44,385	·04; ·036
Lanarkshire bar,... from	64,795	N.		·158
to	51,327	N.		·238

TABLE—*continued.*

Description of Material.	Tenacity in lbs. per Square Inch. Lengthwise.		Crosswise.		Ultimate Extension.
Lancashire bar, ... from	60,110	N.			·269
to	53,775				·216
Swedish bar,......... from	48,933	N.			·264
to	41,27[illegible]				·278
Russian bar, ... from	59,096	N			·153
to	49,564				·133
Bushelled iron from turnings, ...	55,878	N			·166
Hammered scrap,.........	53,420	N.			·248
Angle-iron from various districts, from	61,260	N.			
to	50,056				
Straps from various districts,... from	55,937	N.			·108
to	41,386				·048
Bessemer's iron, cast ingot, ...	41,242	W.			
Bessemer's iron, hammered or rolled, ...	72,643	W.			
Bessemer's iron, boiler plate,..................	68,319	W.			
Yorkshire plates,... from	58,487	N.	55,033	N.	·109; ·059
to	52,000		46,221		·170; ·113
Staffordshire plates, from	56,996	N.	51,251	N.	·04; ·034
to	46,404		44,764		·13; ·059
Staffordshire plates, best-best, charcoal,	45,010	F.	41,420	F.	·05; ·045
Staffordshire plates, best-best, from	59,820	F.	54,820	F.	·05; ·038
to	49,945	F.	46,470	F.	·067; ·04[illegible]
Staffordshire plates, best,	61,280	F.	53,820	F.	·077, ·045
Staffordshire plates, common, ...	50,820	F.	52,825	F.	·05, ·043
Lancashire plates, ...	48,865	F.	45,015	F.	·043; ·028
Lanarkshire plates, from	53,849	N.	48,848	N.	·033; ·014
to	43,433		39,544		·093, ·046
Durham plates, ...	51,245	N.	46,7[illegible]2		·089; ·064

Effects of Reheating and Rolling.

Puddled bar, ...	43,904	C.
The same iron five times piled, reheated and rolled, ...	61,824	C.
The same iron eleven times piled, reheated and rolled, ...	43,904	C.

TABLE—*continued.*

Description of Material.		Tenacity in lbs. per Square Inch. Lengthwise.		Crosswise.		Ultimate Extension.
Strength of Large Forgings.						
Bars cut out of large forgings,	from	47,582	N.	44,578		·231; ·168
	to	43,759	N.	36,824		·205; ·064
Bars cut out of large forgings,..............		33,600	M.			
STEEL AND STEELY IRON. (See also pp. 587 and 793.)						
Cast steel bars, rolled and forged,	from	132,909	N.			·052
	to	92,015	N.			·153
Cast steel bars, rolled and forged,........ ..		130,000	R.			
Blistered steel bars, rolled and forged,...		104,298	N.			·097
Shear steel bars, rolled and forged,...........		118,468	N.			135
Bessemer's steel bars, rolled and forged,...		111,460	N.			·055
Bessemer's steel bars, cast ingots, ...		63,024	W.			
Bessemer's steel bars, hammered or rolled,		152,912	W.			
Spring steel bars, hammered or rolled		72,529	N.			·180
Homogeneous metal bars, rolled, ...		90,647	N.			137
Homogeneous metal bars, rolled, ...		93,000	F.			
Homogeneous metal bars, forged,.........		89.724	N.			·119
Puddled steel bars, rolled and forged,..........	from	71,484	N.			·191
	to	62,768	N.			·091
Puddled steel bars, rolled and forged,...		90,000	F.			
Puddled steel bars, rolled and forged,...		94,752	M.			
Mushet's gun-metal,...		103,400	F.			0·034
Cast steel plates,....	from	96,280	N.	97,150	N.	·057; ·096
	to	75,594	N.	69,082	N.	·198; ·196

TABLE—*continued.*

Description of Material.	Tenacity in lbs. per Square Inch. Lengthwise.		Crosswise.		Ultimate Extension.
Cast steel plates,.... hard	102,900	F.			·031
soft	85,400	F.			·031
Homogeneous metal plates, first quality,	96,280	N.	97,150	N.	·086; ·144
Homogeneous metal plates, second quality,	72,408	N.	73,580	N.	·059; ·032
Puddled steel plates,......... from	102,593	N.	85,365	N.	·028; ·013
to	71,532	N.	67,686	N.	·082; ·057
Puddled steel plates,......	93,600	F.			0·125

In the preceding table the following abbreviations are used for the names of authorities:—

C., Clay; F., Fairbairn; H., Hodgkinson; M., Mallet; Mo., Morin; N.,* Napier & Sons; R., Rennie; T., Telford; W., Wilmot.

The column headed "Ultimate Extension" gives the ratio of the elongation of the piece at the instant of breaking to its original length. It furnishes an index (but a somewhat vague one) to the ductility of the metal, and its consequent safety as a material for resisting shocks. The vagueness arises from the fact, that the elongation of a bar just before breaking consists not so much of a stretching of each particle as of a permanent *drawing out*, with a new arrangement of the particles; and that it bears no fixed proportion, even in the same material, to the original length, being proportionally greater for short than for long bars. It is probable that it depends on the thickness of the piece as well as on its length.

When two numbers separated by a semicolon appear in the column of ultimate extension (thus ·082; ·057), the first denotes the ultimate extension lengthwise, and the second crosswise.

358. **Resilience of Iron and Steel.**—In order to obtain an exact measure of the capacity of a material for resisting shocks *by tension*, the *modulus of elasticity*, as well as the tenacity, must be known; and then the *modulus of resilience* (whose nature and use have been explained in Article 149, p. 227, and Article 173, p. 279) can be computed in *inch-pounds to the cubic inch*, by dividing the *square of the proof tenacity* by the modulus of elasticity. For iron and steel, the proof tenacity may be estimated at *one-third of the ultimate tenacity.*

* The experiments whose extreme results are marked **N.** were conducted for Messrs. R. Napier & Sons by Mr. Kirkaldy. For details, see *Transactions of the Institution of Engineers in Scotland*, 1858-59; also Kirkaldy *On the Strength of Iron and Steel.*

The following table gives some examples of such computations:—

Metal under Tension.	Ultimate Tenacity.	Proof Tenacity.	Modulus of Elasticity.	Modulus of Resilience.
Cast iron— Weak, ...	13,400	4,467	14,000,000	1·425
„ Average, ...	16,500	5,500	17,000,000	1·78
„ Strong, ...	29,000	9,667	22,900,000	4·08
Bar iron—Good average,	60,000	20,000	29,000,000	13·79
Plate iron—Good average,	50,000	16,667	24,000,000?	11·57?
Iron Wire—Good average,	90,000	30,000	25,300,000	35·57
Steel—Soft, ...	90,000	30,000	29,000,000	31·03
„ Hard, ...	132,000	44,000	42,000,000	46·10

To express the power of resisting shocks *by compression*, the resistance to crushing might be substituted in these calculations for the tenacity; but owing to the indirect manner in which fracture by crushing takes place, the result would be of very doubtful accuracy.

359. **Corrosion and Preservation of Iron.**—On this point, see Article 330, p. 462, where some of the best methods of preserving the surface of iron from oxidation have already been mentioned.

Cast iron will often last for a long time without rusting, if care be taken not to injure its skin, which is usually coated with a film of silicate of the protoxide of iron, produced by the action of the sand of the mould on the iron. Chilled surfaces of castings are without this protection, and therefore rust more rapidly.

The corrosion of iron is more rapid when partly wet and partly dry than when wholly immersed in water or wholly exposed to the air. It is accelerated by impurities in water, and especially by the presence of decomposing organic matter, or of free acids. It is also accelerated by the contact of the iron with any metal which is electro-negative relatively to the iron, or in other words, has less affinity for oxygen, or with the rust of the iron itself. If two portions of a mass of iron are in different conditions, so that one has less affinity for oxygen than the other, the contact of the former makes the latter oxidate more rapidly. In general, hard and crystalline iron is less oxidable than ductile and fibrous iron.

Cast iron and steel decompose rapidly in warm or impure sea-water.

Pieces of iron which are kept constantly in a state of vibration oxidate less rapidly than those which are at rest. The comparative rate of corrosion of iron and steel plates has engaged much attention. The subject is fully treated in a paper by Mr. Parker, read before the Iron and Steel Inst., 1881, the general results of which show that steel and iron are very much the same in their rate of loss of strength by corrosion. (See also p. 793.)

SECTION II.—*Of Iron Fastenings.*

360. **Rivets** are made of the most tough and ductile iron or of mild steel. (See also Appendix.) In order that a rivet may connect two or more layers of plates or flat bars firmly, and in order that the shearing stress brought to bear on the rivet by a force tending to pull the plates asunder may be uniformly distributed throughout the sectional area of the rivet, it is essential that the rivet should tightly fit its hole. The longitudinal compression to which the rivet is subjected during the formation of its head, whether by hand or by machinery, tends to produce that result.

The ordinary dimensions of rivets in practice are as follows:—

Diameter of a rivet for plates less than half an inch thick, about double the thickness of the plate.

For plates of half an inch thick and upwards, about once and a-half the thickness of the plate.

Length of a rivet before clenching, measuring from the head = sum of the thickness of the plates to be connected + $2\frac{1}{2}$ × diameter of the rivet.

Inasmuch as the resistance of rivets to shearing is nearly the same with the tenacity of good iron plates (50,000 lbs. per square inch, or thereabouts), the distance apart of the rivets used to connect two pieces of plate iron together is regulated by the rule, that *the joint sectional area of the rivets shall be equal to the sectional area of plate left after punching the rivet holes.* This rule leads to the following algebraical formula (see p. 805):—

Let t denote the thickness of the plate iron,
d, the diameter of a rivet,
n, the number of rows of rivets,

it being understood that the rivets which form a row stand in a line perpendicular to the direction of the tension which tends to pull the plates asunder.

c, the distance from centre to centre of the adjoining rivets in one row; then

$$c = d + \frac{\cdot 7854\, n\, d^2}{t}. \qquad (1.)$$

Each plate is weakened by the rivet holes in the ratio

$$\frac{c - d}{c} = \frac{\cdot 7854\, n\, d}{t + \cdot 7854\, n\, d}; \qquad (2.)$$

In "single-rivetted" joints, $n = 1$; in "double-rivetted" joints

$n = 2$, and the two rows of rivets form a *zig-zag*; in "chain-rivetted" joints, n may have any value greater than 1. A single-rivetted joint is weakened by unequal distribution of the tension in the ratio of 4 : 5.

Suppose that in a chain-rivetted joint the distance c from centre to centre of the rivets is fixed, so as not to weaken the plates below a given limit; then in order to find how many rows of rivets there should be,—in other words, how many rivets there should be in each file,—the following formula may be used:—

$$n = \frac{(c - d)\, t}{\cdot 7854\, d^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.)$$

361. **Pins, Keys, and Wedges.**—These fastenings are, like rivets, themselves exposed to a shearing stress, while they serve to transmit a pull or thrust from one piece of an iron frame to another; and the rule for determining their proper sectional area is the same, with this modification only, that if a pin, key, or wedge is not held perfectly tight in its seat, the shearing stress, instead of being uniformly distributed throughout its sectional area, will be more intense at the central layer of the section than elsewhere, being distributed according to the laws explained in Article 168, p. 266.

The ratio in which the maximum intensity of the shearing stress exceeds its mean intensity is the quantity denoted by q_0 A ÷ F in the table, p. 267; its two most important values in practice being the following:—

For rectangular keys and wedges,............ $\frac{3}{2}$;

For circular or elliptical pins,.. .... $\frac{4}{3}$;

and the sectional area of the fastening is to be increased in this proportion beyond what would be necessary if the stress were uniformly distributed.

In order that a wedge or key may be safe against slipping out of its seat, its angle of obliquity ought not to exceed the angle of repose of iron upon iron, which, to provide for the contingency of the surfaces being greasy, may be taken at about 4°. (Article 110, p. 172.)

362. **Bolts and Screws.**—If a bolt has to withstand a shearing stress, its diameter is to be determined like that of a cylindrical pin. If it has to withstand tension, its diameter is to be determined by having regard to its tenacity. In either case the effective diameter of the bolt is its least diameter; that is, if it has a screw on it, the diameter of the spindle inside the thread.

The projection of the thread is usually *one-half of the pitch;* and the pitch should not in general be greater than *one-fifth of the effective diameter*, and may be considerably less.

In order that the resistance of a screw or screw-bolt to rupture by stripping the thread may be at least equal to its resistance to direct tearing asunder, the length of the nut should be *at least one-half* of the effective diameter of the screw; and it is often in practice considerably greater; for example, once and a-half that diameter.

The head of a bolt is usually about twice the diameter of the spindle, and of a thickness which is usually greater than five-eighths of that diameter.

Section III.—*Of Iron Ties, Struts, and Beams.*

363. **Forms of Iron Bars.**—In designing ordinary structures of wrought iron, it saves time and expense to use iron bars of such forms of cross-section as are usually to be met with in the market. The variety of those forms continually increases with the demand for new shapes; but an engineer should avoid introducing new sections for bars into his designs, except when, by so doing, some important purpose is to be served, or some decided advantage to be gained.

Amongst the most common forms of rolled bars are the following: -

Round iron, with a circular cross-section.

Square iron, with a square cross-section.

Flat iron, with oblong rectangular cross-sections of various forms.

Half-round and convex iron, with one side cylindrical and the other flat.

Angle iron, with cross-sections shaped like an L, of various breadths, depths, and thicknesses.

T-iron, with cross-sections shaped like a T, with a table or flange and a rib of various dimensions.

Double T-iron, or H-iron, or I-shaped iron, with a web and two flanges, of various dimensions. The most common form of railway bars belongs to this class.

Channel iron, which is like a flat bar with flanges projecting from both of its edges, but in one direction only, so that if laid with the flat bar downwards it is like a trough or rectangular channel.

Bulb iron, which is like a flat bar with a cylindrical thickening along one or both edges.

The "Bridge Rail," fig. 229.

The "Barlow Rail," fig. 230.

Bars of a cross-shaped section are sometimes rolled; but the figure is unfavourable to soundness of the iron; and when this form of section is required, it is best to build it. In general, figures for iron bars which cannot be rolled without great distortion of the iron ought to be avoided, unless there are special reasons for using them.

Fig. 229.

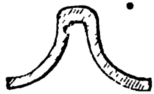


Fig. 230.

Angle bars and plates which exceed an inch in thickness are seldom so sound as those of less thickness. Where greater thicknesses are required, therefore, it is in general advisable to make them by building small thicknesses together.

364. **Iron Ties** ought in almost every case to be of malleable iron, as it has about three times the tenacity of cast iron.

A tie may consist either of one bar, or of several bars side by side, or of wires lying parallel in a bundle or spun into a rope; it may be in one length, or in two or more lengths joined together; if the lengths are numerous and short, they become *links*, and the whole tie a chain.

I. *Plate Iron Ties.*—The best mode of joining two lengths of a plate iron tie is by means of a fish-joint chain-rivetted. In fig. 231 are seen the ends of two lengths of a plate iron tie, meeting at the dotted line; they are connected by means of a fish-piece or *covering plate*, which is chain-rivetted to each of the pieces. The principles according to which the dimensions, number, and arrangement of the rivets are to be determined have been explained in Article 360, p. 515; and it has there also been shown to what extent the *effective sectional area* of the tie is diminished by the rivet holes, so as to be less than the total sectional area.

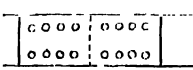


Fig. 231.

When a plate iron tie is built of several layers, they should *break joint* with each other; and at each joint there should be either a covering plate or a pair of covering plates, to transmit that share of the tension which belongs to the layer of plates in which the joint occurs.

II. *Tie-rods* or *Tie-bars* may be round, square, or flat, and may be made fast at the ends by pins passing through round eyes, by wedges driven into oval eyes or slots, or by screws and nuts. The proportions of these fastenings have been considered in Articles 361, 362, pp. 516, 517. Wedges and screws admit of being used to tighten the tie.

When an *eye* is formed on the end of a tie-bar, care should be taken that the sides of the eye are of sufficient strength. The tension is not uniformly distributed in them, being more intense at

the inner side than at the outer. To allow for this, the sectional area may be made one-half greater than would be necessary if the tension were uniformly distributed.

III. *Compound Tie-bars* and *Flat Chains* are made of lengths or links, each consisting of flat bars placed side by side, and connected together by means of eyes and pins, as in the side view, fig. 232, and plan, fig. 232.* Whether it is desired to give stiffness to each individual bar or flexibility to the whole chain, the bars should be on edge. The numbers of bars in a compound link are odd and

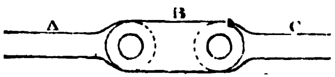


Fig. 232.

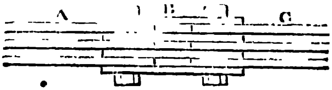


Fig. 232.*

even alternately; thus, in the figure, the links A and C consist of odd numbers of bars, and B of an even number. The dimensions of the pin are to be found as in Article 361, p. 516, taking care to note at how many cross-sections it must give way at once if sheared across; for the stress is distributed amongst those cross-sections. In fig. 232* they are six in number. As to the eyes, see Division II. of this Article above.

IV. In *Oval-linked Chains* it is essential to strength that each link should be prevented from collapsing by a stay or cross-bar, as shown in fig. 233, when it appears by experiment that the tension is uniformly distributed over the cross-sections of the two sides of the link. When the stay is omitted, the strength of the chain is reduced in the proportion of

$$85 : 100, \text{ or nearly } 6 : 7.$$

(Barlow, *On the Strength of Materials*, Article 143.)

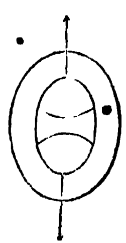


Fig. 233.

V. *Wire Cables* are sometimes made simply of a cylindrical bundle of parallel wires, *served* or bound together by a wire wound round the outside of the bundle. In constructing this sort of tie, great care is necessary in order to distribute the tension equally amongst the wires. The tension is equally distributed, without any special care, in *untwisted wire ropes* in which the wires are spun into strands, and the strands into ropes, without any rotation of individual wires, so that the fibres are all untwisted, and all equally strained. This is the strongest kind of iron tie for its weight; its tenacity, of about 90,000 lbs. per square inch of section, being equal to the weight of 27,000 feet of its own length, or thereabouts.

The best, and perhaps the only safe, mode of *making fast the end* of a wire rope is to make it form a turn or loop round a dead-eye, and splice it into itself: this is the only fastening which is as strong as the rope.

Wire cables require special care to protect them against oxidation. (See p. 798.)

VI. *Welded Ties.*—Iron ties have been lengthened by scarfing the ends of the two pieces together, heating them to a welding heat by a gas flame, and welding them together by an intense pressure. Data are wanting to determine precisely the strength of this sort of joint. In an experiment on the bursting of a cylindrical plate iron welded retort, the tenacity of the welded joint was found to be

30,750 lbs. per square inch;

or probably about 3-5ths of the tenacity of the plate iron

VII. In *proving Iron Ties*, they may safely be loaded with one-half of the instantaneous breaking load, without risk of permanent injury, the testing load being only applied once; although frequent application of the same load would at last break the tie. (See p. 800.)

365. **Cast Iron Struts and Pillars.**—Cast iron, from its great resistance to crushing, is peculiarly well suited for struts and pillars, especially those of moderate length. The best form for a cast iron strut or pillar containing a given quantity of material is that of a hollow cylinder. The laws of the strength of such pillars have already been fully explained in Article 158, pp. 236 and 237. The thickness of metal in them is seldom less than one-twelfth of the diameter.

Another form of cross-section commonly adopted for cast iron struts is that of a cross, fig. 234. The strength of such struts may be computed approximately by putting for the co-efficient a in equations 4 and 5 of Article 158, p. 237, a value greater than its value for a hollow cylinder, in the same proportion as a cross-shaped bar is more flexible than a hollow cylindrical tube of the same diameter and sectional area; that is to say, in the proportion of 3 to 1 nearly.

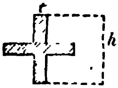


Fig. 234.

By similar reasoning, it appears that in the case of a hollow square cast iron strut, whose *diagonal* is equal to the diameter of the cylinder, the co-efficient a is to be increased in the ratio of 3 to 2.

Hence we have the following approximate formulæ for the *crushing load* of cast iron struts in lbs. *per square inch of sectional*

Cross; diameter from end to end of a pair of arms $= h$; .... $80{,}000 \div 1 + \frac{3\,l^2}{800\,h^2}$;

Hollow square; diagonal $= h$; .... $80{,}000 \div 1 + \frac{3\,l^2}{1600\,h^2}$;

Hollow cylinder; diameter $= h$; .... $80{,}000 \div 1 + \frac{l^2}{800\,h^2}$.

The preceding formulæ refer to the case in which the struts are fixed in direction at the ends. When they are hinged at the ends, the second term of each divisor is to be made four times as great.

In order to give lateral stiffness to a flat-ended pillar, its ends should spread so as to form a capital and base, whose abutting surfaces should be "faced" in the lathe, or planed, to make them exactly plane and perpendicular to the axis of the pillar. For the same reason, when a cast iron pillar consists of two or more lengths, the ends of those lengths should be made truly plane and perpendicular to the axis of the pillar by the same process, so that they may abut firmly and equally against each other; and they should be fastened together by at least four bolts passing through projecting flanges.

366. **Wrought Iron Struts and Pillars.**—The principles of the strength of wrought iron struts have been explained in Article 158, pp. 237, 238. It appears from the formulæ deduced by Mr. Gordon from Mr. Hodgkinson's experiments, that while cast iron is the better material for a pillar whose length does not exceed a certain limit as compared with its diameter, wrought iron is the better material when the length exceeds that limit. For pillars with fixed ends, that limit, according to the formulæ, is about 26 times the diameter; for those with hinged ends, about 13 times; but from the nature of the calculation, those results must be regarded as roughly approximate only. (See p. 795.)

In order to stiffen wrought iron struts, they are made of various forms in cross-section, such as the angle iron, T iron, double T iron, channel iron, &c., already described. A very convenient form of cross-section is that of a cross. It is in general built by rivetting bars of simpler forms together; thus, it may be made up of two T-irons rivetted back to back, or four angle irons rivetted back to back, or (as in fig. 235) one flat bar A A, two narrower flat bars, B, B, and four angle irons, all rivetted together. This last form is that of the strut-diagonals of the Warren girders in the Crumlin Viaduct. A double T-shaped strut may either be a single bar, or may be built in a manner which will be described in treating of beams. The *Barlow Rail* (fig. 230, p. 518) is also a good form for struts.

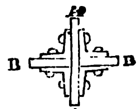


Fig. 235.

The stiffest form for a wrought iron strut is that of a *cell*, that is to say, a built tube, which may be cylindrical, rectangular, or triangular. Fig. 236 is a cross-section of a rectangular cell, with four plate iron sides connected together by angle irons and rivets. *Fig.* 237 is a triangular cell, running along the upper edge of a plate iron beam.

Fig. 238 shows a simple form of cross-section for a strut, being

Fig. 236.

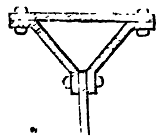


Fig. 237.

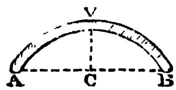


Fig. 238.

ment of a circle. This might be further stiffened by rivetting a pair of angle irons along its edges.

When a wrought iron strut is *hinged* at the ends, that generally takes place by its abutting at each end against a cylindrical pin, by which it is connected with some other piece of the framework, in the manner already described for tie-bars. To *fix* its ends in direction, as it seldom has large abutting surfaces, it is in general necessary to fasten it to the adjoining pieces of the structure by several bolts or rivets.

To insure the stiffness of a *built* strut, the bars of which it is built should break joint, like the layers of a built iron tie. The abutment of successive lengths against each other should be firm and equable; to insure which, every bar should have its ends made exactly plane and exactly perpendicular to its length. This is best done by a machine consisting of a pair of circular saws on one axis, at a clear distance apart equal to the intended length of the bars when cut: a bar being placed parallel to the axis, and moved towards it, has its two ends sawn off at once, in planes perpendicular to its length.

Mr. Gordon's formula for the ultimate strength of wrought iron struts, deduced from Mr. Hodgkinson's experiments, may be expressed as follows, P being the load, S the sectional area, l the length, and h the thickness:—

$$\frac{P}{S} = 36{,}000 \div \left(1 + \frac{a\,l^2}{h^2}\right) \ldots\ldots\ldots\ldots\ldots (1.)$$

where a has the value $\frac{1}{3{,}000}$ for struts fixed in direction at the ends, and of a solid rectangular section, h being the least dimension.

For other forms of cross-section, an approximate rule has already been given, to the effect that h is to be considered as representing the least dimension of a triangle or rectangle circumscribed about the bar; but in many cases it may be more satisfactory to take into account the least "radius of gyration" of the cross-section, as in the last article; and for that purpose the formula may be put in the following shape, r denoting that radius; that is to say, r^2 is the mean of the squares of the distances of the particles of the cross-section from a neutral axis traversing its centre of gravity in that direction which makes r^2 least:—

$$\frac{P}{S} = 36{,}000 \div \left(1 + \frac{l^2}{36{,}000\, r^2}\right) \quad \ldots\ldots\ldots\ldots (2.)$$

For hinged ends, put 9,000 instead of 36,000 in the divisor.

The following are values of r^2 for different figures:—

I.	Solid rectangle; least dimension $= h$;	$h^2 \div 12.$
II.	Thin square cell; side $= h$;...	$h^2 \div 6.$
III.	Thin rectangular cell; breadth b; depth h;......	$\frac{h^2}{12} \cdot \frac{h+3b}{h+b}$
IV.	Thin triangular cell on the edge of a plate (fig. 237); base of triangle $= h$;	$h^2 \div 12.$
V	Solid cylinder; diameter $= h$;	$h^2 \div 16.$
VI.	Thin cylindrical cell; diameter $= h$;................	$h^2 \div 8.$
VII.	Angle iron of equal ribs; breadth of each $= b$;	$b^2 \div 24.$
VIII.	Angle iron of unequal ribs; greater b, less h;	$b^2 h^2 \div 12\,(b^2 + h^2).$
IX.	Cross of equal arms;	$h^2 \div 24.$
X.	H-iron; breadth of flanges b; their joint area A; area of web B;...........	$\frac{b^2}{12} \cdot \frac{A}{A+B}$
XI.	Channel iron; depth of flanges + ½ thickness of web, h; area of web B; of flanges A;.............	$h^2 \cdot \left\{ \frac{A}{12(A+B)} + \frac{AB}{4(A+B)^2} \right\}.$
XII.	Barlow rail; cross-section composed of two quadrants of radius R, measured to middle of thickness, connected by a table of sectional area = joint area of quadrants × ·273;	$R^2 \div 7$ nearly.

XIII. Pair of Barlow rails as above, rivetted base to base; ...	}	$\cdot 393\ R^2$.
XIV. Circular segment of radius R and length 2 R θ;	}	$\left\{\frac{1}{2}+\frac{\cos\theta\sin\theta}{2\theta}-\frac{\sin^2\theta}{\theta^2}\right\} R^2$.

367. **Cast Iron Beams.**—For the principles which are applicable to cast iron beams in common with beams in general, see Articles 169 to 179 A, pp. 230 to 296. The peculiar properties of cast iron as to strength, which have to be considered in designing beams, have been stated in Article 164, p. 257; Article 166, p. 261; Article 167, pp. 263 to 265; and in Article 353, pp. 499 to 502; and the precautions to be observed in designing these, as well as other castings, have been explained in Article 354, p. 502.

The most common and useful forms of cross-section for cast iron beams are the inverted ⊥-shaped (fig. 239), the trough-shaped (fig. 240), and the double T-shaped (fig. 241).

As to the transverse resistance of the ⊥-shaped section, see Article 163, Example VIII., p. 254. As to the proportionate area

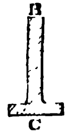


Fig. 239.

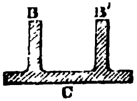


Fig. 240.

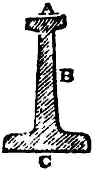


Fig. 241.

of flange and web which makes the tendency to break by crushing at B and tearing at C equal, see Article 164, Case I., p. 257; also the example in the same page. The same formulæ and example are applicable to trough-shaped beams, taking the two vertical ribs, B, B′, to be equivalent to one rib of the same depth and double the thickness.

The thickness of the horizontal and vertical parts of these girders should be equal, or nearly equal, for the reason stated in Article 354, p. 503.

The double T-shaped beam is in general made of a figure introduced by Mr. Hodgkinson, with a view to making the strength equal above and below. As to the proportions of that section, see Article 164, Case II., p. 257; also the example in the same page. See also Hodgkinson *On the Strength of Cast Iron.*

In order to make the stretched table C large enough as compared with the compressed table A, it is necessary to make the former considerably thicker than the latter. This is reconciled with the

rule as to avoiding abrupt changes of thickness in castings, by making the vertical web B of the same thickness with A at the top, and of a gradually increasing thickness towards the bottom, where it is nearly as thick as C.

Transverse ribs or feathers on cast iron beams are to be avoided, as forming lodgments for air-bubbles, and as tending to cause cracks in cooling.

Open-work in the vertical web is also to be avoided, partly for the same reasons, and partly because it too much diminishes the resistance to distortion by the shearing action of the load.

The various "*forms of equal strength*" in longitudinal sections, already described in Article 165, pp. 259, 260, are more easily executed in cast iron than in any other material, and are often employed in practice, especially those shown in figs. 141, 142, 143, 144, and 145.

The forms of horizontal section shown in figs. 142 and 144 are applied to the flanges or tables of double T-shaped girders of uniform depth.

In supporting a cast iron beam, provision must be made at one end for its expansion and contraction by heat and cold, which take place at the rate of about ·00111 for the 180° Fahr. between the ordinary freezing and boiling points of water, or ·0000062 nearly per degree of Fahrenheit's scale.

368. **Lengthened and Trussed Cast Iron Beams.**—It is seldom advisable to use a cast iron beam of a span so great that it cannot be cast in one length; but should it nevertheless be determined to do so, the following principles are to be observed in the construction of each junction.

Above the neutral axis the ends of the pieces should be true planes, abutting closely and equably against each other, and exactly perpendicular to the axis of the beam.

Below the neutral axis the pieces are to be connected by means of transverse flanges and wrought iron bolts, which will thus, at the joint, perform the duty of the lower or stretched table of the beam; and the total sectional area of those bolts should be such as to make, *not* their tenacity, but their *proportionate elongation by a given tension*, the same with that of the cast iron table for which they are a substitute. This condition will be approximately fulfilled by making the sectional area of the bolts in all about *one-half* of that of the cast iron table; when their tenacity will be more than sufficient

Care should be taken so to arrange the bolts that the mean of the squares of their distances from the neutral axis of the section shall *not be less* than the corresponding quantity for the cast iron table whose duty they are to perform.

The same principles are to be followed in designing that sort of

trussed cast iron beam in which a pair of wrought iron tie-rods are substituted for the whole or part of the lower table.

369. **Plain Wrought Iron Beams.**—For the principles which are applicable to wrought iron beams in common with beams in general, see Articles 169 to 179 A, pp. 230 to 296.

The most common and useful forms of section for wrought iron beams that are rolled in one piece are the T-shaped, and the I-shaped or double T-shaped, of which latter form fig. 242 is an example.

As to the resistance of cross-sections of those figures to cross-breaking, see Article 254, Examples VIII. and IX., pp. 254 to 256. As to the mode of fixing the proportions of such sections in order that they may be of equal strength against crushing and tearing, see Article 164, Cases III. and IV., p. 258, and the example in the same page; these being the cases applicable to a material in which the tenacity (denoted by f_b in the formulæ) is greater than the resistance to crushing (denoted by f_a).

Fig. 242.

A plain wrought iron beam usually gives way under a transverse load by the compressed flange bending sideways; for that flange is in general so narrow, as compared with its length, that its condition is analogous to that of a long wrought iron strut. (See Article 158, p. 237.) The co-efficient f_a, therefore, which is the modulus of resistance of that flange, is not in general a constant quantity, but is less as the flange becomes narrower in comparison with the span of the beam.

From a reduction of the experiments of Fairbairn on wrought iron beams, given in his works, *On the Application of Iron to Building Purposes*, and *Useful Information for Engineers*, first series, it appears that the modulus in question may be computed with sufficient accuracy by the following formula, in which

l denotes the span of the beam, and
b, the breadth of its compressed flange:—

$$f_a \text{ (in lbs. on the square inch)} = \frac{36{,}000}{1 + \dfrac{l^2}{5{,}000\, b^2}} \dots\dots (1.)$$

The following table shows the result of applying this formula to variously proportioned beams, and of substituting its results in equations 10 and 11 of Article 164, p. 258; the value of the tenacity f_b being assumed to be 60,000 lbs. per square inch in each case. A_1 denotes the sectional area of the compressed flange; A_2 that of the vertical web; A_3 that of the stretched flange; h' the

depth from centre to centre of the two flanges; M_0 the breaking moment in inch-lbs. :—

	f_b	$\frac{l}{b}$	f_a		$M_0 \div h'$.
I.	60,000	10	35,294	$\cdot 35\ A_2 + 1 \cdot 7\ A_3$	$14,118\ A_2 + 60,000\ A_3$
II.	60,000	20	33,333	$\cdot 4\ A_2 + 1 \cdot 8\ A_3$	$14,444\ A_2 + 60,000\ A_3$
III.	60,000	30	30,509	$\cdot 48\ A_2 + 1 \cdot 97\ A_3$	$14,916\ A_2 + 60,000\ A_3$
IV.	60,000	40	27,273	$\cdot 6\ A_2 + 2 \cdot 2\ A_3$	$15,455\ A_2 + 60,000\ A_3$
V.	60,000	50	24,000	$\cdot 75\ A_2 + 2 \cdot 5\ A_3$	$16,000\ A_2 + 60,000\ A_3$

The preceding results are made applicable to the T-shaped section simply by making $A_3 = 0$.

In cases in which a beam is liable to be strained alternately in either direction, the section is to be made similar above and below, so that $A_1 = A_3$; the beam tends to give way in every case by lateral bending of the flange which is compressed for the time, and f_a is the modulus of rupture; and the expression for the breaking moment assumes the simplified form,

$$M_0 = f_a h' \left(\frac{A_2}{6} + A_1 \right) \ldots\ldots\ldots\ldots\ldots\ldots (2.)$$

The *expansion* of wrought iron beams by heat is very slightly greater than that of cast iron beams, being about ·0012 for the 180° between the ordinary freezing and boiling points of water, or about ·0000067 per degree of Fahrenheit.

370. **Plate and Box Beams.**—This class of iron or steel beams comprises various cross-sections, built of plates and bars rivetted together in various ways, but all virtually equivalent to double T-shaped sections, and having their strength dependent on the same principles. The following are examples :—

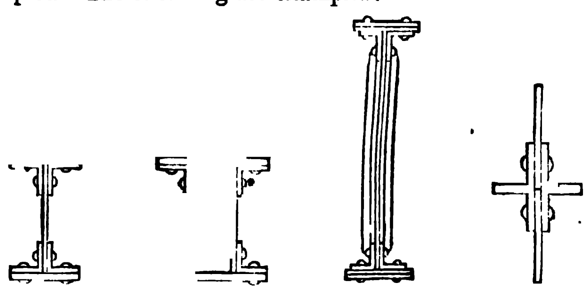


Fig. 243. Fig. 244. Fig. 245. Fig. 246.

Fig. 243 is a plate beam having a single plate for the vertical web, while each of the horizontal ribs or flanges consists of a flat

bar and a pair of angle irons, rivetted to each other and to the vertical web.

Fig. 244 is a "box beam," in which there is a double vertical web. The advantage of this construction is to give additional breadth, and therefore additional lateral stiffness and additional strength for resisting thrust, to the compressed table or flange.

Fig. 245 is a plate beam of greater dimensions than fig. 243. The horizontal ribs or flanges contain more than one layer of flat bars, and the web, which consists of plates with their largest dimension vertical, is stiffened by vertical T-iron ribs at the joints of those plates, as shown in the horizontal section, fig. 246.

To give still greater stiffness and strength to the upper or compressed horizontal rib, it is sometimes a cylindrical tube or "cell;" sometimes a rectangular cell, as in fig. 236, p. 522; sometimes a triangular cell, rivetted to the upper edge of the vertical web, as in fig. 237, p. 522; and in some cases a line of plates bent into an inverted segmental trough, as shown in the cross-section, fig. 238, p. 522, has been made fast at its summit V, by angle-irons and rivets, to the upper edge of the vertical web.

In fixing the dimensions of the parts, and computing the strength, of beams of this class, the rules of the preceding article are all applicable, having regard to the following special principles:—

I. The several tiers or layers of pieces of which the beam is built should break joint as much as possible.

II. *Upper Horizontal Rib.*—The several lengths of the pieces composing the upper horizontal rib should abut closely and truly against each other, having end surfaces made exactly perpendicular to the axis of the beam, as already described for wrought iron struts in Article 366, p. 522. In using equation 1 of Article 369, p. 526, to compute the modulus of rupture by crushing, f_a, the following are the divisors by which $\frac{l^2}{b^2}$ is to be divided:—

For a flat upper horizontal rib, or a triangular cell, ...	5,000
For a square cell, ...	10,000
For a cylindrical cell or an inverted semicircular trough (diameter $= b$),	7,500
For an inverted segmental trough, subtending the angle 2θ to radius unity,	$\frac{7,500}{\sin^2\theta}\left(1 - \frac{\sin 2\theta}{2\theta}\right)$.

III. *Lower Horizontal Rib.*—The several lengths of plates or bars of which the lower horizontal rib consists are to be connected with each other by covering-plates and rivets as prescribed for wrought iron ties in Article 364, p. 518; and the symbol A_2 in the formulæ

of Article 369, and of the other articles there referred to, is to be understood to stand, not for the total sectional area of the lower rib, but only for the *effective sectional area* left after making the proper deduction for rivet-holes, according to the principles explained in Article 364, p. 518, and Article 360, p. 515.

For the best plate iron, the value of the modulus of tenacity, f_b, is on an average about 50,000 lbs. per square inch.* The following are the results of substituting that value for 60,000 in the computations of the table in Article 369, p. 527:—

	f_b	l For a Flat Upper Rib.	f_a	A_1	$M_0 - h'$
I.	50,000	10	35,294	$\cdot21\ A_2 + 1\cdot41\ A_3$	$10{,}784\ A_2 + 50{,}000\ A_3$
II.	50,000	20	33,333	$\cdot25\ A_2 + 1\cdot5\ \ A_3$	$11{,}111\ A_2 + 50{,}000\ A_3$
III.	50,000	30	30,509	$\cdot32\ A_2 + 1\cdot64\ A_3$	$11{,}584\ A_2 + 50{,}000\ A_3$
IV.	50,000	40	27,273	$\cdot41\ A_2 + 1\cdot83\ A_3$	$12{,}121\ A_2 + 50{,}000\ A_3$
V.	50,000	50	24,000	$\cdot54\ A_2 + 2\cdot08\ A_3$	$12{,}667\ A_2 + 50{,}000\ A_3$

IV. *Vertical Web.*—The thickness of the vertical web is seldom made less than $\frac{3}{8}$ inch, and, except in the largest beams, is in general more than sufficient to resist the shearing stress. In those beams in which it becomes necessary to attend specially to the power of the vertical web to resist the shearing action of the load, the amount of that shearing action is to be computed for a sufficient number of cross-sections by the formulæ of Article 161, Case IX., pp. 247, 248, 249, and its greatest intensity, for an assumed thickness of web, by the formula of Article 168, Case VIII., p. 267. (In many cases, however, it is sufficiently accurate to assume the shearing stress to be entirely borne by the vertical web, and uniformly distributed throughout its section.) It is then to be considered that the shearing stress at the neutral axis is equivalent to a pull and a thrust of equal intensity inclined opposite ways at 45°, and that the vertical web tends to give way by buckling under the thrust; so that its ultimate resistance in lbs. per square inch is given by the following expression:—

$$\frac{36{,}000}{1 + \dfrac{s^2}{3{,}000\ t^2}} \qquad (1.)$$

in which t is the thickness of the plates of the web, and s the distance measured along a line inclined at 45° to the horizon, between two of its vertical stiffening ribs; or, if it has no such ribs, between the upper and lower horizontal ribs. The intensity of the shearing

* About 66,000 lbs. is the corresponding average value for steel plates.

action of the working load should not exceed one-sixth of the resistance given by the above formula.

V. *Longitudinal Variations of Section.*—Inasmuch as the bending moment of the load diminishes from the middle of the beam towards the ends, and the shearing force from the ends towards the middle, according to principles stated in Article 161, pp. 245 to 249, the transverse sections of the horizontal ribs may be diminished from the middle towards the ends, and that of the vertical web from the ends towards the middle, so as to make the resistance to bending and shearing respectively vary according to the same law.

VI. *Vertical Ribs.*—Each vertical rib is to be considered either as a suspending-piece from which a portion of the load hangs, or as a pillar on which a portion of the load lies, according as the load is hung from or supported upon the beam; and its transverse section must be made sufficient for the duty so thrown upon it, according to the principles of Article 364, p. 518, or Article 366, p. 521, as the case may be; and regard must be had to the fact, that a large rolling load, such as that upon one of the wheels of a locomotive engine, may be concentrated upon one vertical rib.

Above each of the *points of support*, the vertical ribs must either be placed closer or made larger, so that they may be jointly capable of safely bearing, as pillars, the entire share of the load which rests on that point of support.

A pair of vertical T-iron ribs rivetted back to back through the web-plates may be held to act as a pillar of cross-shaped section. (Article 366, Case IX., p. 523.)

(*Note as to Diagonal Ribs.*—It is obvious that the best position of the stiffening ribs would be diagonal, sloping upwards from the ends of the beam towards the middle at angles of 45°; but this would involve inconvenience and expense in workmanship, and would cause the plates for the web to be cut into awkward and complex figures).

VII. *Rivets.*—The principles which regulate the number and dimensions of the rivets that connect the lengths of the stretched horizontal rib together have been sufficiently explained in the passages referred to in Division III. of this Article.

The rivets which connect one division of the web with an adjoining vertical rib should be capable of withstanding safely the greatest shearing action of the load at the joint in question.

The shearing action on the rivets which connect one of the horizontal ribs with a given division of the web is to the vertical shearing action of the load at the middle of that division very nearly as the length of the division in question is to its depth.

VIII. *Camber.*—In order that a built wrought iron beam may

become nearly straight under its working load, it should be constructed in such a manner that, if unloaded, it would have a "camber" or upward convexity equal to the anticipated working deflection.

Owing to the yielding of the joints, it is found that, in computing the deflection of plate girders, when first loaded, a smaller modulus of elasticity ought to be taken than for continuous iron bars. Its value in lbs. per square inch is about 17,500,000, or two-thirds of the value for a continuous bar, so that the deflection is about one-half greater. But the part of that deflection due to the yielding of the joints is permanent; so that after the joints have "come to their bearing," the modulus of elasticity becomes the same as for a continuous bar.

With a cross-section of equal strength, the working deflection is as follows:—

$$v_1 = \frac{n'' (f_a' + f_b') l^2}{4 E h}. \qquad (2.)$$

Taking 6 as the factor of safety, we may make, with sufficient accuracy for the present purpose,

$$f_a' + f_b' = (f_a + f_b) \div 6 = 84,000 \div 6 = 14,000.$$

For an uniformly-loaded beam, with an uniform cross-section, $n'' = \frac{5}{12}$; if the cross-section varies along the beam so as to be of uniform strength and uniform depth, $n'' = \frac{1}{2}$. Assuming the latter to be nearly the case, we have

$$\text{working deflection } v_1 = \frac{14,000}{8 \times 17,500,000} \cdot \frac{l^2}{h} = \frac{l^2}{10,000\, h}; \quad (3.)$$

or *a third proportional to the depth and the hundredth part of the span.*

For example, an ordinary proportion is $l = 15\,h$; then $v_1 = \frac{3\,l}{2000}$.

It is advisable to drill the rivet holes, especially when steel is used, and if several plates are to be united the holes should be drilled together.

371. **Great Tubular Girders.**—This term is applied to hollow plate iron girders so large that the traffic of a bridge can pass through the interior.

Fig. 247 is a skeleton diagram of a cross-section of the class of tubular girder used in the Conway and Britannia bridges, in which the top and bottom are *cellular*, consisting of plates so disposed as to form rows of square or nearly square cells, like that in fig. 236,

p. 522. In order that these cells may be sufficiently stiff, the width of each of them should not exceed *thirty times the thickness of the plates* of which they are made. The joints of the cells are connected and stiffened by means of covering plates outside as well as angle irons within. The breadth is fifteen feet.

The two vertical webs, or sides, B, exactly resemble the vertical

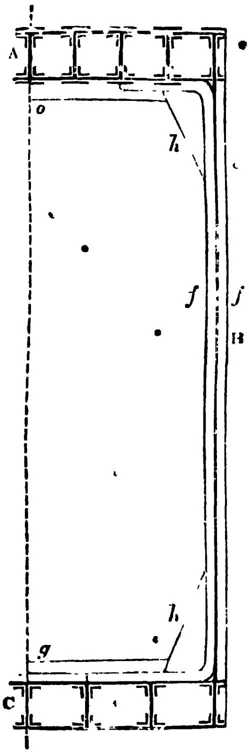


Fig. 247.

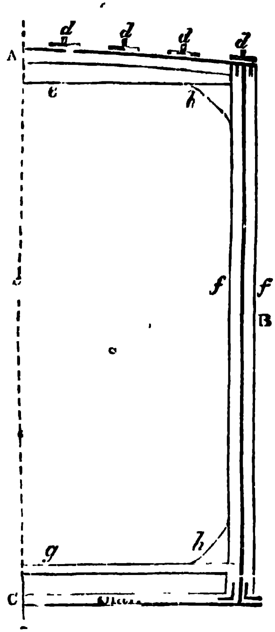


Fig. 248.

web already described in Division IV. of the last article, being composed of plates set up on end, and connected by means of pairs of vertical T-iron ribs, *f*, *f*. The horizontal joints of the side plates are made fast by covering strips. The lateral steadiness of the connection between the horizontal tables and the vertical webs is assisted by means of gussets, *h*, *h*, and horizontal prolongations of

the inner T-iron ribs. The top and bottom are further stiffened by transverse ribs, *e*, *g*, one at every third set of vertical ribs.

Fig. 248 shows one-half of a kind of cross-section for a great tubular girder 16 feet broad, in which the top and bottom consist of one or more layers of plates rivetted close together, and stiffened by means of projecting ribs, instead of by a cellular structure. The girders of the Victoria Bridge over the St. Lawrence were of this class. A is the upper table; it is slightly arched, having a radius of curvature equal to about six times its breadth, and is stiffened by the longitudinal T-iron ribs *d*, *d*, *d*, *d*, about 2 feet 3 inches apart, and by the transverse ribs *e*, about 7 feet apart. B is one of the sides, with vertical T-iron ribs in pairs, *f*, *f*, about $3\frac{1}{2}$ feet apart. C is the bottom, consisting of a sufficient thickness of plates, with covering strips; it is stiffened, so far as it needs stiffening, by its connection with the transverse joints *g* of the platform, which are double T-shaped plate beams, one at every 7 feet; *h*, *h*, are stiffening gussets. There is now a double line of rails and openwork girders in place of the former tubular girders.

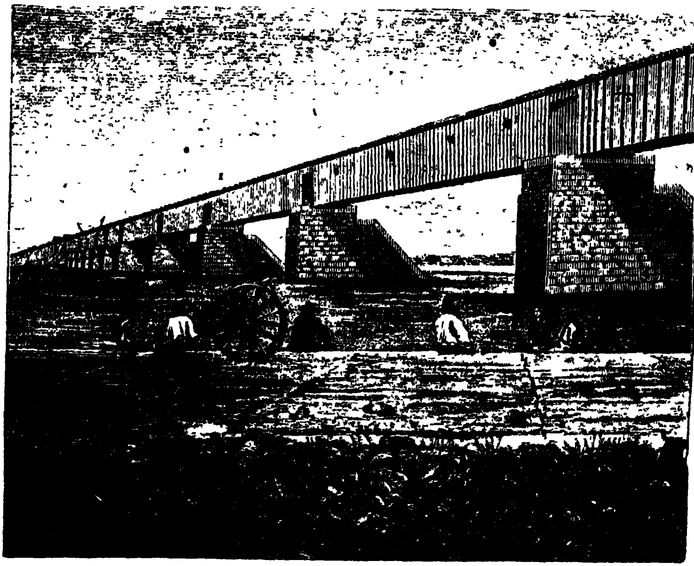


Fig. 249.—[The Victoria Bridge, from a Photograph.]

The whole of the principles of construction and strength stated in the preceding article for plate and box-beams are applicable to great

tubular girders. In applying to them the principle of Division VI. of that Article, p. 530, so far as it relates to the strength and stiffness required for the vertical ribs at the points of support, it may be found necessary greatly to enlarge those ribs, and to give them the form of double T-shaped plate girders standing on end, and tapering from below upwards.

To illustrate the relative proportions of the parts of which a tubular girder is composed, the following statement shows those proportions for the tubes of the Conway Bridge:—

The quantities of iron in the top and bottom are nearly equal; for though the top, being compressed, has a larger *effective* section than the bottom, which is stretched, the total section of the bottom is increased so as to be nearly equal to that of the top, by the greater dimensions of the covering plates required at the joints.

The two sides together contain nearly the same quantity of iron with the top.

The distribution amongst the various parts is as follows:—

	Top. Per cent.	Sides. Per cent.	Bottom. Per cent.	
Plates,............................	61·0	51·3	61·1	
Angle iron and T-iron,...........	29·3	37·2	14·9	
Covers,............................	3·8	5·4	19·7	
Rivet-heads,......................	5·9	6·1	4·3	
	100·0	100·0	100·0	
Proportion per cent. of effective to total section,.....	87·7	43·0	72·2	Without deducting rivet-holes
Proportion per cent. of effective to total section of bottom, deducting *one-seventh* for rivet-holes,......			61.9	

(See Baker, *Long and Short Span Railway Bridges*; Fidler, *Bridge Construction*; Anglin, *The Design of Structures.*)

372. In the **Erection of Iron Girders**, three methods may be followed: a girder may be built on the ground and lifted to its place, it may be moved endwise upon rollers on to its piers; or it may be built in position on a scaffold. The first method was adopted with the girders of the Britannia Bridge, each of which was floated on pontoons to a position between the piers directly below its permanent position; the faces of the piers having recesses to admit the ends of the girder. It was then lifted by means of

chains, hanging from the cross-heads of the plungers of a pair of hydraulic presses on the top of the piers, through a height equal to the stroke of the plungers (6 feet). As the beam rose, the recesses below its ends were built up with brickwork, which formed a pair of temporary supports for it while the plungers were lowered and the chains shortened, in order to raise it through the height of another stroke, and so on. The girders of the Victoria Bridge were built upon a scaffolding in their final position, all the pieces of which they were made having been shaped and punched in England.

The method of moving endwise on rollers on to the piers is best adapted to girders that are continuous over two or more spans; and such girders may require during the process to be temporarily stiffened by means of masts and stay-chains.

In order to give a girder put up span by span the property of *continuity* over its piers, the following method has been practised:—Suppose two lengths of the girder to have been erected, and to be still discontinuous over the pier where they meet. Each of those lengths is bent by its own weight; and their adjoining ends, instead of standing in parallel vertical planes, lean away from each other. The further end of one of the lengths is now tilted up, by means of a hydraulic press, a lifting jack, or some such suitable machine, until the two adjoining ends meet accurately; when they are made fast to each other by fishing, bolting, rivetting, or other suitable means of connection. The further end of the girder that has been tilted up is now lowered into its proper place; and the same process is followed for each joint where continuity over a pier is required.

As to the effect of continuity over the piers upon the strength and stiffness of a girder, see Article 178, p. 287, and especially Method II. of that Article, pp. 288 to 292.

The continuity of a girder must not be carried throughout a greater length than is consistent with a proper provision for its expansion and contraction by changes of temperature. An iron girder can have only one fixed support; all the rest must be on roller beds or slides; and in the case of a girder continuous over piers the best position for the fixed point of support is near the middle of its length. The largest continuous tubular girders erected are those of the Britannia Bridge; they are 1,511 feet in length, and rest on three piers and two abutments, forming four spans; they have a fixed support on the central pier, and rest on rollers at the other four points of support, so that the length of metal which expands and contracts at each side of the fixed support is $755\frac{1}{2}$ feet.

From what has been stated respecting the mode of connecting the lengths of a continuous girder, it is obvious that, previous to

the making of the connection, each length bends under its own weight as a separate girder, and that the whole of its top should be stiffened to resist compression. After the connection has been effected, the top of each girder assumes a state of tension, and the bottom a state of compression, from the piers to the points of *contrary flexure* (p. 291, equation 10). Hence both the top and the bottom of a girder which is to be continuous over the piers are to be stiffened by means of cells or ribs, so as to be capable of resisting either compression or tension. Such is the case in the Britannia Bridge, where the girders are cellular both above and below. (See the authorities cited in p. 534.)

It has already been shown in Article 178, Method II., equations 7 and 8, pp. 289, 290, that if w be the fixed load, and w' the rolling proof load (being twice the ordinary rolling load) per unit of length, the moments of flexure are respectively,

$$\text{over a pier,} -M_1 = \frac{2w + w'}{24} \cdot l^2; \quad \ldots\ldots(1.)$$

$$\text{in the middle of a span, } M_0 = \frac{w + 2w'}{24} l^2; \quad \ldots\ldots(2.)$$

$$\text{the sum of which, or } \frac{w + w'}{8} \cdot l^2, \quad \ldots\ldots(3.)$$

is simply the moment of flexure in the middle of a separate girder. The effect of the operation, then, already described, by which a girder is made continuous over the piers, consists in relieving the middle of each span of the girder of bending action to the amount denoted by the expression (1), and transferring that amount of bending action to the parts over the piers. If, as is the case in tubular bridges of the largest class, the rolling load is less than the fixed load, (1) is greater than (2); but the most advantageous method of employing the strength of the material, is to make the bending actions at mid-span and at each pier equal to each other, each of them being one-half of the expression (3); that is to say,

$$\frac{w + w'}{16} \cdot l^2. \quad \ldots\ldots(4.)$$

To effect this result, an *imperfect continuity* is to be produced in the following manner:—

Observe the angular opening between the end surfaces of a pair of lengths of the girder as they lean from each other before being connected; denote it by θ, then compute the following quantity:—

$$4 w + 2 w' \cdot \theta; \qquad \ldots (5.)$$

and let this be the angle between the faces of a wedge-shaped filling piece to be introduced into the opening between the ends of the lengths of girder before connecting them, so that the opening may be reduced in the ratio of the expression (4) to the expression (1). Then tilt up the further end of one of the lengths until the joints fit, and connect them.

If the rolling load is to be neglected in making this adjustment, the expression (5) becomes simply $\theta - 4$, so that the angular opening is

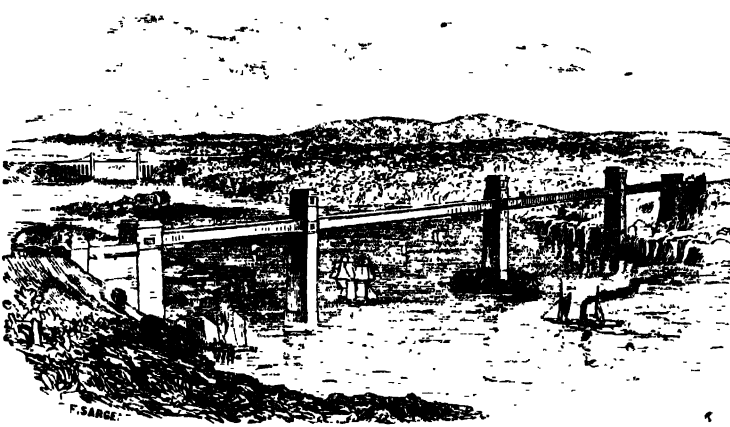


Fig. 250.—[The Britannia Bridge.]

to be reduced to 3-4ths, and the further end of one of the lengths tilted up to 3-4ths of the height which would have been necessary for perfect continuity. The result is, that the moments of flexure at mid-span and at the piers become equal to each other for a fixed load. This was the method followed at the junction over the central pier of the Britannia Bridge.

373. **Effect of Wind on Tubular Girders.**—The pressure of the wind against one side of a tubular girder acts like an uniformly distributed load tending to bend it sideways, and producing a bending moment and maximum stress whose values are as follows:—

Let w'' be the pressure per lineal inch of girder, found by multiplying the intensity of the pressure of the wind by the depth of the side of the girder

l, the span;
b, the breadth, from centre to centre of the vertical sides.
A_1 and A_3, as before, the sectional area of the top and bottom; and A_2, the joint sectional area of the sides.
Then the greatest bending moment is,

$$M = \frac{w'' l^2}{8} \text{ at the middle for a separate single span girder; (1.)}$$

$$M = \frac{w'' l^2}{12} \text{ at each pier for a girder continuous over the piers; (2.)}$$

and the greatest stress is

$$p_1 = M_1 \div b\left(\frac{A_2}{2} + \frac{A_1 + A_3}{6}\right) \ldots\ldots\ldots\ldots\ldots (3.)$$

The greatest pressure of wind ever observed in Britain* was 55 lbs. on the square foot, = ·382 lb. on the square inch. (See p. 222*b*.)

374. **Plain Arched Iron Ribs** are usually made of cast iron, but wrought iron is sometimes employed. The present article is confined to those iron arches which have not, or do not depend for their stiffness upon, diagonal bracing in their spandrils, so that the disfigurement of each rib is resisted either by its own stiffness alone, or by that stiffness combined with the stiffness of a horizontal girder directly above the rib.

The whole theory of the action of a load on an arched rib has already been given in Article 180, pp. 296 to 314, with the exception of some cases which have come to the author's knowledge since that part of this work was in type, and which will be treated of in this and in some subsequent articles.

Cases in which the arched rib is so braced by means of the spandril-framing that a special theory is required for them, will be treated of in the next section.

Reference will now be made to those parts of preceding articles where the formulæ to be used in computing the strength of arched ribs are to be found.

The usual form of section is the I or double T-shape, with equal flanges above and below, the thickness of the web being equal, or nearly equal. The depth (denoted by h in the formulæ) is not generally to be computed by means of a formula, but is to be found, either by a series of trials, or by adopting an empirical rule, such as making it from 1-40th to 1-60th of the span. The ratio q, to be used in the formulæ for strength and stiffness, is to be computed for

* By the late Dr. Nichol, at Glasgow Observatory.

such a section by means of the expression in Case IX. of Article 179, p. 295.

The neutral axis of the rib should be a parabola; for which, in arches of small rise as compared with the span, an arc of a circle may be substituted without material detriment to the stability of the arch.

The rib may be made of *uniform stiffness* by increasing the sectional area from the crown to the springing in the proportion of the secant of the inclination, as explained in Article 180, Problem II. When the rib is not of uniform stiffness, but of *uniform section*, the computation by means of the formulæ gives the area of section *at the crown of a rib of uniform stiffness of the same strength*, and this must be augmented in the proportion of the secant of the inclination of the neutral axis at the springing to radius, in order to obtain the *uniform area of section* required for the proposed rib of uniform section.

CASE I.—When the rib has flat ends firmly bedded against immovable skew-backs (as is usually the case with cast iron arches), the case is that of Article 180, Problem IV.; and the first step in the calculation is to compute the quantity denoted by B, by means of equation 30, p. 305, viz.,—

$$B = \frac{45\, q m' h^2}{4\, k^2} \cdot \left(1 + \frac{16\, k^2}{3\, l^2}\right).$$

Should the skew-backs, through the yielding of piers or abutments, spread asunder when the arch is loaded, the case is that of Problem V., and B is to be computed by means of equation 40, p. 308. In using this last equation, a sectional area, A_1, has to be assumed.

To allow for the straining effects of rise and fall of temperature, proceed as follows:—Let t denote the greatest probable deviation of the temperature from that at which the bridge is to be erected; E, the modulus of elasticity of the material; p_0, the intended mean intensity of the greatest thrust at the crown of the rib; and e, the co-efficient of expansion, whose value is

·0000067 per degree of Fahrenheit, or
·0000120 per centigrade degree,

then, in computing B, by means of equation 30 or equation 40, the following additional term is to be introduced into the factor within the brackets:—

$$\mp \frac{e\, t\, E}{p_0}; \qquad (1.)$$

the sign $\left\{\mp\right.$ being used according as t denotes $\left\{\begin{matrix}\text{rise}\\\text{fall}\end{matrix}\right\}$ of temperature.

The mean intensity of thrust p_0 may be unknown; in which case a provisional calculation of the horizontal thrust and area of section must be made without allowing for the effects of change of temperature, in order to obtain an approximate value of p_0.

When B has been computed, the next step is to compute, by the formulæ 36, p. 307, the proportions r_1 and r'_1 of the span which must be loaded with a rolling load, in order to make the thrust and tension respectively the greatest possible.

Should the sectional area be fixed, the greatest thrust p_1 and the greatest tension p'_1, are then to be computed by means of equations 37 and 38 respectively, p. 307.

Should the sectional area have to be fixed by computation, transpose, in these equations, the symbols A and p; they then become formulæ for computing the sectional area, if the greatest safe working thrust and tension respectively be put for p_1 and p'_1; and the *greater* of the two values of A_1 is to be adopted for the area at the crown of a rib of uniform stiffness.

To find the total horizontal thrust H when the stress is greatest, use equation 31, p. 306. The quantity p_0 in the expression (1) above has for its value $H \div A_1$.

The greatest total horizontal thrust is found by making $r = 1$.

The approximate formulæ, 37 A, 38 A, p. 307, and 31 B, 33 B, 37 B, 38 B, p. 308, may be used in the cases there explained.

CASE II.—When the rib is fixed at the crown to a *horizontal girder*, see Problem VIII., pp. 313, 314.

CASE III.—The rib may be vertically *hinged at the ends*, by having them rounded, and supported by hollow cylindrical bearings, so that they resemble trunnions or journals. This case falls under Problem VI.; and the first step in the calculation is to compute the value of the quantity C by means of equation 51, p. 310.

To allow for the effect of changes of temperature, introduce into the factor of C within the brackets, the expression (1) of this Article, already explained.

The most severe stress occurs very nearly when one half of the span is loaded. Under that condition,

To find the total horizontal thrust H, use equation 52 A, p. 311;

To find the greatest moment of flexure M′, use equation 57 A, p. 312;

To find the greatest intensity of stress if the sectional area has been fixed, use equation 58, p. 311;

To find the required sectional area at the crown, make p_1 = the greatest safe working intensity of thrust; and use the following formula:—

$$A_1 = \frac{1}{p_1}\left(\frac{M'}{q\,h} + H\right). \qquad (2.)$$

In Cases I., II., and III., *to find the greatest deflection*, see Problem VII., pp. 312, 313.

CASE IV.—*Rib hinged at the crown and at the ends.*—The hinging of the rib at the crown, as well as at the ends, has been proposed by M. Mantion (*Annales des Ponts et Chaussées*, 1861), but has never yet been executed. This mode of construction would have the great advantage of annulling the straining effect both of changes of temperature, and of the yielding of the piers. The formulæ for computing the greatest stress in this case are deducible from those of Problem VI., pp. 311, 312, by making $C = 0$; and they are as follows:—

Let l be the span, k, the rise, of the neutral line in inches;
w_0, the fixed load per lineal horizontal inch;
w, the rolling load per lineal horizontal inch;

then the greatest intensity of stress occurs when one half of the rib is loaded with the rolling load; and in that condition the total horizontal thrust is,

$$H = \frac{l^2}{8\,k}\left(w_0 + \frac{w}{2}\right); \qquad (3.)$$

the greatest moment of flexure, which acts downwards on the loaded half and upwards on the unloaded half of the rib, is

$$M' = \frac{l^2\,w}{64}; \qquad (4.)$$

the greatest intensity of thrust occurs at the outer flange of the loaded and the inner flange of the unloaded half of the rib, and has the following value:—

$$p_1 = \frac{1}{A_1}\left(\frac{M'}{q\,h} + H\right) = \frac{l^2}{8\,A_1}\left\{\frac{w}{8\,q\,h} + \frac{1}{k}\left(w_0 + \frac{w}{2}\right)\right\}; \ldots (5.)$$

and the greatest intensity of tension, if any, occurs at the inner

flange of the loaded and outer flange of the unloaded rib, and has the following value:—

$$p'_1 = \frac{1}{A_1}\left(\frac{M'}{q\,h} - H\right) = \frac{l^2}{8\,A_p}\left\{\frac{w}{8\,q\,h} - \frac{1}{k}\left(w_0 + \frac{w}{2}\right)\right\} \ldots (6.)$$

To proportion the depth of the rib h to its rise k, so that the greatest tension may bear any given ratio to the greatest thrust, make,

$$\frac{q\,h}{k} = \frac{w}{8\,w_0 + 4\,w} \cdot \frac{p_1 - p'_1}{p_1 + p'_1}; \ldots \ldots (7.)$$

(for the value of q, as before, see Article 179, Case XI., p. 295.)

The *greatest total horizontal thrust* occurs when the rib is loaded over its whole span; and its amount is,

$$H_1 = \frac{l^2}{8\,k}(w_0 + w). \ldots (8.)$$

In many of the older examples of cast iron arched bridges, the ribs consist of a large number of small cast iron open-work panelled frames, acting as voussoirs, and bolted, dowelled, or otherwise connected together; but this mode of construction is deficient in strength and stability; and in later and better examples the ribs are made in as few and as long pieces as is practicable, and these are made to abut firmly and accurately against each other at planed surfaces, and are connected by means of transverse flanges and bolts. In cast iron arches of moderate size each rib usually consists of two lengths only, bolted together at the crown. In Southwark Bridge the ribs consist of pieces of 20 feet in length, whose ends abut, not directly against each other, but against transverse plates, which serve to bind the several parallel ribs of the bridge together crosswise, and through which the flanges of the lengths of the ribs are bolted together. In the Westminster Bridge each rib consists of five pieces, the side pieces being of cast iron, and the middle piece of wrought iron.

The subject of iron arched ribs will be further considered in treating of *braced iron arches* in the next section.

The Tower Bridge across the River Thames below London Bridge is a structure consisting partly of suspension and partly of girder design. The essential features are two large towers in the river from which chains pass to the shore where they are anchored to girders embedded in concrete. A high level girder foot bridge, near the top of the towers, about 140 feet above high-water mark, serves to connect the towers, and through the medium of stairs and

hydraulic hoists serves to maintain the cross river communication when the low level roadway is opened for the passage of vessels.

This part of the bridge consists of two great platforms free to rise and fall like the halves of an ordinary canal bridge, being hinged at their ends next the great towers. The length of the bridge is 940 feet, which is mainly made up as follows:—Two shore spans, each of 270 feet; one centre or opening span of 200 feet, and the widths of the two central towers which are each 70 feet.

The width is 60 feet. The towers are of steelwork resting on piers of brickwork on a foundation of Portland cement concrete.

The chains are formed of girder pattern so as to ensure rigidity under unequal loading Provision is made for movements due to expansion and contraction, both at the points of connection of the suspension spans with the fixed masonry and elsewhere. The opening span is of special interest as the movable halves of the span each weigh fully 1,000 tons. Transverse girders serve to divide the main girders of the bascules or opening span upon which buckled plates are fixed. The roadway is paved with pine blocks grouted with asphalt.

The centre of gravity of each of these movable spans rests upon a solid steel forging, 21 inches diameter, serving as a centre of motion.

The steel used was limited to 27 tons tensile strength, and the working stress allowed in the calculations was 6 tons per square inch.

Hydraulic machinery is used for the working of the bascules and for the passenger hoists in the towers. The water pressure, 700 lbs., is obtained from two compound engines of 360 I.H.P. each.

(See *Minutes Proceedings Inst. C.E.*, vols. cxiii., cxxi., and cxxvii.)

The East River Bridge, New York, is on the suspension principle, having a main span of 1,595 feet 6 inches. There are 4 cables, each of which contains 5,296 *parallel* galvanised steel oil-coated wires, wrapped together, making a cylinder 15¾ inches diameter. The ultimate strength of such a cable is estimated at 12,200 tons. The height of towers above high water is 278 feet, giving a clear height at centre of span, at 90° Fah., of 135 feet.

The Forth Bridge, erected at Queensferry, has two main spans of 1,700 feet each, and is constructed of steel, on the bracket system, having a central intermediate girder, over 300 feet long, in each main span, resting on the ends of the triangular-shaped

brackets or cantilevers. The height of bridge at centre of span above high water is 150 feet (See also p. 801.)

The Douro Bridge, near Oporto, is the largest example of an arch-rib bridge yet constructed, the central span being 520 feet, and the weight of iron in the arch alone 504 tons.

The Niagara Cantilever Railway Bridge is 910 feet long, the two cantilevers being 395 feet each, and the intermediate span 120 feet. The towers are constructed of braced work, 130 feet high, and rest on masonry piers 39 feet high. The depth of the Cantilever truss over the towers is 56 feet; at shore end 21 feet, and at river end 26 feet. The trusses are 18 feet apart. Factor of safety 5. Steel and iron are used in the construction.

A bridge of somewhat peculiar construction has recently been constructed at Pittsburgh, in America. The main feature consists in two large trusses of 360 feet span each. These trusses are somewhat of the Bowstring pattern, only the lower member is curved, both top and bottom members being arcs of circles. The depth of trusses at centre is 50 feet. The upper member consists of four steel plates, about 1 foot deep, stiffened by 4 × 4 angles. The lower member is composed of bars, about 28 feet long. The whole truss is divided into 11 bays or panels by vertical pieces and cross-bracing. The ends of each truss are fastened to the top of a pillar 29 feet above top of piers. The roadway is carried by suspenders from the main trusses, the lower part of which just touches the roadway at centre of span. Steel plates were mainly used in the construction, as also steel rivets and pins for connections. The quality of the steel used was, for compression members, ultimate strength from 80,000 to 90,000 lbs. per square inch, with an elastic limit of from 50,000 to 55,000 lbs. per square inch. For tension members, ultimate strength from 70,000 to 80,000 lbs. per square inch, with an elastic limit of 45,000 to 50,000 lbs. per square inch. From experiment it appeared that the loss of strength round the rivet holes, due to punching, was partly restored by the annealing effect of the hot rivets, so that any special reaming of the holes was dispensed with.*

SECTION IV.—*Of Iron Frames.*

375. **Iron Platforms.**—A platform in which timber planking is supported by iron girders, or girders and joists, requires no remarks beyond those which have already been made in Article 336, pp. 465 to 468, regard being had to the difference of the material of

* See *Engineering*, Mar. 14, 1884.

the joists. As to the weight of platforms with their loads, see p. 466.

As to the distribution of the load of the platform of a railway bridge amongst the girders, see Article 341, pp. 475 to 477.

Iron may be used as the covering of a platform in various forms.

I. The *Barlow Rail* is a good form of section for supporting very heavy loads. (See fig. 230, p. 516; also Article 366, Example XII., p. 523.) When proportioned as directed in the example cited (that is to say, the sectional area of the table = the joint area of the quadrantal wings × ·273), it has the following properties:—

Let R be the radius of the quadrantal wings measured to the middle of their thickness.

t, their thickness, then;

$$\left.\begin{array}{l}\text{Sectional area of Barlow Rail} = 4\,\mathrm{R}\,t \text{ very nearly;}\\ \text{The neutral axis is nearly at the middle of the depth; and}\\ \text{Breaking moment} = \frac{2}{7} f_a\,\mathrm{R} \times \text{area} = \frac{8}{7} f_a\,t\,\mathrm{R}^2 \text{ nearly;}\end{array}\right\}(1.)$$

f_a being the modulus of rupture by crushing or buckling of the top table, or probably from 30,000 to 35,000 lbs. per square inch.

II. *Corrugated Iron* should be so supported that the bending action of the load takes place in a plane parallel to the ridges and furrows. Iron laths should be rivetted across the ridges and furrows to prevent them from spreading. These may be at distances apart equal to about twice the breadth of the corrugations.

Let *b* denote the breadth of a sheet of corrugated iron, *h* the depth from ridge to furrow; *t*, the *virtual thickness* in inches = weight in lbs. per square foot of horizontal projection ÷ 40, then,

$$\text{breaking moment} = \frac{4}{15} f\,h\,b\,t. \ldots\ldots\ldots\ldots\ldots(2.)$$

Least modulus of rupture, $f = 34,200$ lbs. on the square inch, by Mr. Hart's experiments. (See p. 799.)

III. *Bending Moment of the Load on a Plate.*—When a rectangular plate is supported on two parallel edges, the bending moment exerted by a load placed upon it is the same as that exerted by the same load on a beam of the same span.

When a rectangular plate is firmly supported at all its four edges by joists and girders, the bending moment is diminished. If the plate is square, the bending moments exerted in planes parallel to its two dimensions are equal to each other; if it is oblong, the greatest bending moment is exerted in a plane parallel to the

breadth, or lesser dimension of the plate, the tendency being to split it lengthwise at the middle of its breadth.

The following formulæ are founded on a theory which is only approximately true, but which nevertheless may be considered to involve no error of practical importance:—

Let W denote the total load.
l, the length of the plate, between the supports of its ends.
b, its breadth, between the supports of its side edges.
M, the greatest bending moment.

CASE I.—Square plate, load uniformly distributed;

$$M = \frac{W b}{16} \quad \text{.........................(3.)}$$

CASE II.—Square plate, load collected in the centre;

$$M = \frac{3 W b}{16} \quad \text{.........................(4.)}$$

CASE III.—Oblong plate, load uniformly distributed;

$$M = \frac{W l^4 b}{8 (l^4 + b^4)} \quad \text{.........................(5.)}$$

CASE IV.—Oblong plate, load collected in the centre; l less than 1·19 b;

$$M = \frac{3 W l^4 b}{8 (l^4 + b^4)} \quad \text{.........................(6.)}$$

CASE V.—Oblong plate, load collected in the centre, l equal to or greater than 1·19 b;

$$M = \frac{W b}{4}; \quad \text{.........................(7.)}$$

being the same as for a plate supported at the *side edges only*.

CASE VI.—Circular plate, of the diameter b, supported all round the edge, load uniformly distributed:

$$M = \frac{W b}{6 \pi} = ·053 \text{ W } b. \quad \text{.....................(8.)}$$

CASE VII.—Circular plate, load collected in the centre;

$$M = \frac{W b}{2 \pi} = ·159 \text{ W } b. \quad \text{.....................(9.)}$$

IV. *Cast Iron Flooring Plates.*—The breaking moment of these plates is to be made greater than the bending moment of the working load in the ratio of a suitable factor of safety (such as *six*). They should be strengthened by means of vertical ribs or feathers on the upper side; and then the moment of resistance may be computed as for a trough shaped or ⊥-shaped girder. (Article 367, p. 521; Article 163, pp. 264, 265.)

V. *Buckled Wrought Iron Plates* (the invention of Mr. Mallet) are plates of various figures (usually square or oblong), having a slight convexity in the middle, and a flat rim round the edge, called the "fillet;" and are the best form yet devised for the iron covering of a platform. They are usually placed so that the convex part is compressed, and the flat fillet stretched, and when they give way under an excessive load, it is usually by the crushing or crippling of the convex part.

Let l be the length of that section of a buckled plate at which the greatest bending moment is exerted (according to the principle stated at the beginning of Division III. of this Article, p. 513); h, the depth of curvature at the centre of the plate; t, its thickness, all in inches. Then the moment of resistance is nearly

$$\frac{4}{15} f_a\, l\, h\, t; \qquad (10.)$$

f_a being a modulus of rupture by crushing or crippling the plate at its crown or most convex part. From published results of experiment, it appears that for a plate 36 inches square, including the fillet, which is 2 inches broad, with a curvature of 1·75 inch, and $\frac{1}{4}$ inch thick, made fast all round the edges, the crushing load distributed over the plate is about 18 tons; whence, according to Case I.,

$$f_a = 21{,}600 \text{ lbs. per square inch nearly.}$$

This co-efficient, like that expressing the resistance of wrought iron struts to crushing (see Article 366, p. 522, equation 1), probably varies with the proportion of the thickness of the plate to its breadth, having for its maximum value 36,000; but sufficient experiments have not yet been published to show the law of its variation precisely. According to the table of safe loads for buckled plates 3 feet square, published by the inventor, the safe load varies nearly as the square of the thickness; this would make the co-efficient f_a vary nearly in the simple ratio of the thickness for plates of equal breadth, and of proportionate thicknesses, within the limits of those mentioned in the table, which are ·048 inch and ·375 inch. The factor of safety adopted being 4 for a steady load, and 6 for a moving load, the safe loads given in the table for a

plate 3 feet square, $\frac{1}{4}$ inch thick, and with 1·75 inch of curvature, are 4·5 tons for a steady load, and 3 tons for a moving load. The buckled plates used by Mr. Page for the platform of the New Westminster Bridge measure 84 inches by 36, with a curvature of $3\frac{1}{2}$ inches, and thickness of $\frac{1}{4}$ inch; they bear 17 tons on the centre without giving way. According to Case V., this corresponds to a maximum thrust at the convex part of 17,920 lbs. per square inch.

The square form of buckled plates, supported and fastened at all the four edges, is the most favourable to strength.

376. **Iron Roofs.**—An iron roof may either be made entirely of iron, or the framework may be of iron and the covering of some other material. As to the construction and weight of various sorts of covering for roofs, see Article 337, p. 468. To the sorts of covering there described there may now be added *buckled iron plates*, already described in the last article; the thicknesses suited for roofing being from 1-20th to 1 10th of an inch.

The framework of iron roofs consists of parts analogous to those already described in treating of the framework of timber roofs in Articles 338, 339, pp. 469 to 475, with the exception, that in roofs covered with sheet iron, whether plain, corrugated, or buckled, the "common rafters" are unnecessary; the covering being supported on horizontal T-iron or angle iron bars, which act as laths or as purlins, and which are themselves supported on the principal rafters. Those principal rafters, and the trusses to which they belong, are placed at regular distances of from 2 feet 6 inches to 7 feet apart; the average distance is about 5 feet.

The general designs of those frames or trusses are analogous to those used in timber roofs; and in the computation of the thrusts and pulls along the several pieces, the same formulæ are applicable. (See Article 339, pp. 469 to 475.) In iron roof trusses, however, there is seldom a *tie-beam*: the principal tie being usually a single rod, supported at one or more points, and having no transverse load except its own weight between the supported points.

To the examples of roof trusses given in Article 339, may be added the following, which illustrates a kind of secondary trussing peculiar to iron roofs as distinguished from timber roofs:—1 2 3 is

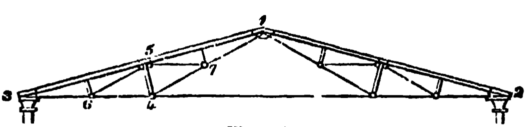


Fig. 251.

the *primary truss*, consisting of the two rafters 1 2, 1 3, and the principal tie-rod 2 3. To find the stresses on its pieces, conceive

half the weight of a division of the roof to be concentrated at the point 1, and proceed as in Article 339, Case I., p. 469.

1 3 4 is a *secondary truss*, supporting the middle point 5 of the rafter by the aid of the strut-brace 4 5. Conceive *one quarter* of the weight of a division of the roof to be concentrated at 5, and proceed as in Article 119, figs. 102, 103, p. 182.

5 6 3, 1 7 5, are *smaller secondary trusses*, similar to 1 3 4, but of half the dimensions, and each sustaining *one-eighth* of the weight of a division of the roof.

The points of support of the rafter may thus be multiplied to any required extent.

The total or resultant stress on each portion of each bar of the truss is to be determined by the aid of the principle of Article 121, p. 184.

Thus the pull on the middle division of the great tie-rod is simply that due to the primary truss, 1 2 3. The pull on 4 7 is simply that due to the secondary truss 1 4 3. The pulls on 5 7 and 5 6 are simply those due to the smaller secondary trusses 1 5 7, 5 6 3. The pull on 1 7 is the sum of those due to the trusses 1 4 3 and 1 7 5. The pull on 6 4 is the sum of those due to the trusses 1 2 3 and 1 4 3. The pull on 6 3 is the sum of those due to the trusses 1 2 3, 1 4 3, and 5 6 3. The thrust on each of the four divisions of the rafter 1 3 is the sum of three thrusts, due to the primary truss, the larger secondary truss, and one of the smaller secondary trusses respectively.

As to the effect of *cambering the principal tie* by bracing it up to the top of the truss, see Article 119, fig. 100, p. 181.

In the *construction of iron roof-trusses* the rafters are usually made of T-shaped or H-shaped iron bars, and the struts of T-iron or angle iron bars, or any convenient form for resisting thrust. As to the strength of struts of these and other figures, see Article 366, pp. 521 to 524. The divisions of a rafter, and also the struts, may be considered as *hinged at the ends*. For the struts, cast iron is sometimes employed. (See Article 365, p. 520.) The smaller ties are usually round or square rods; the larger ties are sometimes flat bars set on edge. The foot of a rafter may be connected with the end of the great tie-bar by a gib and key traversing an oblong slot (Article 361, p. 516); or the foot of the rafter may abut into a cast iron shoe, to which the tie-rod may be fastened by a key, a pin, or a screw and nut. (Article 362, p. 516.) The oblique and vertical ties, or suspending-pieces, generally have *jaws* or forks at their upper ends, where they are hung from the rafters by means of pins, and screws at their lower ends, where they are connected with the struts and with the great tie-bar by means of pinching nuts. A central vertical suspending-rod is

called a "king-bolt;" lateral vertical suspending-rods are called "queen-bolts."

A roof may have arched iron ribs instead of rafters. As to their strength, see Article 374, pp. 537 to 542. As to the stress on a semicircular rib, see the formulæ for such ribs when made of timber, Article 345, pp. 481, 482.

A simple and light roof for moderate spans is made by using bent sheets of corrugated iron so as to act at once as a covering and an arch, the thrust at the foot being resisted by horizontal tie-rods. As to the strength of corrugated iron, see Article 375, p. 543.

377. **Iron Braced Girders—General Design.**—Iron trusses or braced girders are analogous, in their figure and in the action of the load upon them, to the timber "bridge trusses" already described in Article 341, p. 475, and the same formulæ are to a great extent applicable to both. The chief differences are, that pieces which act alternately as struts and as ties are more frequently found in iron than in timber trusses; and that in iron trusses figures frequently occur which resemble those of timber trusses inverted, so that the ties become struts and the struts ties.

For the distribution of the load amongst a set of parallel bridge-girders, see pp. 475 to 477.

The following are examples of the general designs of iron braced girders:—

I. *Triangular Truss.*—(See fig. 252.) This exactly resembles the triangular timber truss, fig. 207, p. 470, inverted; B B being a strut, supported in the middle by the strut D, and the tie-rods A and C. The stress on each of its pieces may be computed by means of the formulæ 1, p. 470, substituting thrust for tension, and tension for thrust.

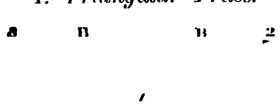


Fig. 252.

If each of the divisions, B, B, of the horizontal strut acts also as a *beam*, supporting a distributed load, the greatest intensity of thrust amongst its particles is to be computed by the formula (already given for arched ribs),

$$p = \frac{1}{A}\left(\frac{M}{q\,h} + H\right); \quad \ldots\ldots\ldots\ldots\ldots\ldots (1.)$$

in which H is the horizontal thrust, computed as in p. 470;

M, the bending moment;

A, the sectional area of the strut B B;

h, its depth;

q, a factor depending on its figure, as to which, see Article 179, p. 295

By transposing p and A in equation 1, above, it becomes a formula for computing the required sectional area, p being made equal to the greatest working thrust per square inch.

II. *Trapezoidal Truss.*—(See fig. 253.) This resembles the trapezoidal timber truss, fig. 209, p. 470, inverted; B B B being a horizontal strut, supported by vertical struts K, K; A, F, and C, are the principal ties; G, G, tie-braces, which act only when the points 5 and 6 are unequally loaded.

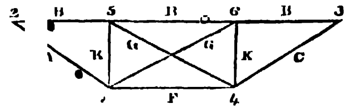


Fig. 253.

The greatest stress on the principal pieces takes place when both points 5 and 6 are fully loaded.

Let W denote the greatest load on each of these points (including *one-quarter* of the weight of the truss itself);
- c, the half-span of the truss;
- x, the distance of each of the points 5 and 6 from the middle of the span;
- k, the depth of the truss, measured from the centre of the horizontal strut B B B to the centre of the horizontal tie F;
- H, the total thrust along B B B, and total tension along F;
- T, the total tension on each of the inclined ties, A, C;

then

$$\mathrm{H} = \mathrm{W}(c-x) \div k;\ \mathrm{T} = \sqrt{(\mathrm{H}^2 + \mathrm{W}^2)} \ldots\ldots\ldots (2.)$$

To find the greatest amount of tension, S, on each of the diagonal braces, G, G, let W′ be the *greatest excess* of the load on either of the points, 5, 6, above the load on the other point; then (as in equation 4, p. 477),

$$\mathrm{S} = \left(\frac{\mathrm{G}}{k} + \mathrm{W}' \frac{c-x}{2ck}\right) \cdot \sqrt{4x^2 + k^2} \ldots\ldots\ldots (3.)$$

G being the weight of one of the braces.

The greatest thrust on each of the vertical struts K, K, is given by the expression,

$$\mathrm{V} = \mathrm{W}' \cdot \frac{c-x}{2c} + \frac{\mathrm{B}}{4} + \mathrm{K} + \mathrm{G}; \qquad (3.\mathrm{A}.)$$

in which B denotes the weight of the horizontal strut B B B, and K that of the upright itself.

III. *Zig-zag Truss, or Warren Girder.*—This girder consists of upper and lower horizontal booms, the former of which acts as a strut, and the latter as a tie, in resisting the bending action of the

load; of a series of diagonal braces forming a zig-zag, which resist the shearing action of the load by thrust and tension alternately; and in some cases of a series of vertical suspending-rods to hang cross-joists from the upper row of joints, such as 1, 3, 5, N – 1, in fig. 255.

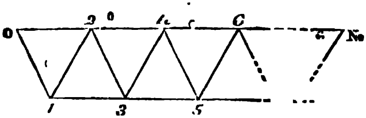


Fig. 254.

Fig. 254 represents the general design of a Warren girder suited for supporting a platform above it, at the points marked with the even numbers, 2, 4, 6, &c. (as in the Crumlin Viaduct, fig. 228, p. 494). Fig. 255 represents the general design of a Warren girder suited for supporting a series of cross-joists below it, hung from all the joints, 1, 2, 3, 4, 5, 6, &c., N – 1.

The actions of the load on this girder are computed by the method already explained in Article 160, pp. 230 to 243, as applied to a beam loaded at detached points. When every joint is

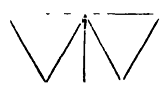


Fig. 255.

equally loaded (as in fig. 255), the formula for the bending moment at any cross-section is that of Article 161, Case VIII., p. 247. In computing the shearing force, regard must be had to the action of a travelling load, as explained in Article 161, Case IX., pp. 247, 248.

The most convenient method of computing the stress on each piece of a Warren girder is by means of a series of additions and subtractions, general formulæ being only used to check the accuracy of the results.

The first step is to number all the joints of the girder, as in the figures, designating one of the points of support as 0, and the other as N; N being the number (always even) of equal horizontal divisions into which those joints divide the span. Let n denote the number affixed to any particular joint;

l, the span of the girder;

k, its depth, from centre to centre of the horizontal booms;

s, the length of each diagonal brace;

$$\left(= \sqrt{k^2 + \frac{l^2}{N^2}}\right);$$

F_n, the shearing action at a cross-section between the joints n

and $n+1$; thus, F_0 is the shearing action between 0 and 1; F_1, between 1 and 2, &c.

H_n, the pull or thrust, as the case may be, upon that piece of a horizontal boom which lies between the joints $n-1$ and $n+1$; that is, opposite the joint n; for example, H_1 is the tension on 0 2 in fig. 255, or the thrust on 0 2 in fig. 254; H_2 is the thrust on 1 3 in fig. 255, or the tension on 1 3 in fig. 254; and so on.

The most severe bending action at each cross-section takes place when the girder is loaded over the whole span; the most severe shearing action at any given cross-section, when the larger segment of the span is loaded and the shorter unloaded; therefore the former supposition must be made in computing the stress on the horizontal booms, and the latter in computing the stress on the diagonal braces.

Two cases may be distinguished; that of fig. 255, in which the load is applied at every joint, and that of fig. 254, in which the load is applied at the joints marked with even numbers only. The former, though the more complex in construction, is the simpler in calculation, and is therefore taken first.

Case I.—*Each joint loaded.* Let the fixed part of the load on each joint be w, the rolling part w'; so that

$$W = (w+w')(N-1), \qquad (4.)$$

is the full load of the girder.

To find the Horizontal Stresses.

Compute the supporting pressure (F_0) at each of the points 0 and N by taking half the full load; that is to say,

$$F_0 = \frac{W}{2} = (w+w') \cdot \frac{N-1}{2}. \qquad (5.)$$

Then compute the first term, and by successive subtractions of the quantity $\frac{l}{N k}(w+w')$, all the other terms, of the following series, which is that of the shearing actions, each multiplied by the ratio of the length of one horizontal division of the span to the depth of the girder.

$$\begin{aligned} &\frac{l}{N k} F_0; \\ &\frac{l}{N k} F_1 = \frac{l}{N k} F_0 - \frac{l}{N k}(w+w'); \\ &\frac{l}{N k} F_2 = \frac{l}{N k} F_1 - \frac{l}{N k}(w+w'); \\ &\quad \&c. = \&c. \end{aligned} \qquad (6.)$$

Then the series of horizontal stresses on the several divisions of the booms are to be computed by successive *additions*, as follows:—

$$\left.\begin{aligned} H_1 &= \frac{l}{N k} F_0; \\ H_2 &= H_1 + \frac{l}{N k} F_1; \\ H_3 &= H_2 + \frac{l}{N k} F_2; \\ \&c. &= \&c. \end{aligned}\right\} \quad \text{.....................(7.)}$$

The test of the accuracy of this series of calculations is, that for the middle horizontal piece, whose number is $N \div 2$, it should give a result agreeing with that of the following formula:—

$$H_{\frac{N}{2}} = \frac{N l}{8 k} (w + w'). \quad \text{.....................(8.)}$$

This is the maximum value of H, which has equal values for pieces equally distant from the middle piece.

The value of H for any particular piece whose number is n may be tested, if required, by the following formula:—

$$H_n = \frac{l}{N k} (w + w') \cdot \frac{n (N - n)}{2} \quad \text{.................(9.)}$$

To find the Diagonal Stresses due to the fixed part of the Load.

Let the stress produced by the fixed part of the load on the diagonal brace which lies between the joints n and $n + 1$ be denoted by T_n. This will be a pull or a thrust alternately, according as the brace in question slopes downwards or upwards towards the middle of the span. The values of this stress are computed by a series of subtractions of the constant difference $\frac{s w}{k}$ as follows:—

$$\left.\begin{aligned} T_0 &= \frac{s w}{k} \cdot \frac{N - 1}{2}; \\ T_1 &= T_0 - \frac{s w}{k}; \\ T_2 &= T_1 - \frac{s w}{k}; \\ \&c. &= \&c. \end{aligned}\right\} \quad \text{.....................(10.)}$$

and the accuracy of the calculations is tested by the rule, that for the braces adjoining the middle joint of the girder, the result should be $s\,w \div 2\,k$.

To find the Diagonal Stresses due to the rolling part of the Load.

These stresses are proportional to the series of "triangular numbers," 0, 1, 3, 6, 10, &c., which result from the successive addition of the natural numbers, 1, 2, 3, 4, &c.; and they are to be computed as follows:—By successive additions of the common difference $\frac{s\,w'}{\mathrm{N}\,k}$, form the following arithmetical series, containing $\mathrm{N}-1$ terms,

$$\frac{s\,w'}{\mathrm{N}\,k};\ 2\frac{s\,w'}{\mathrm{N}\,k};\ 3\frac{s\,w'}{\mathrm{N}\,k};\ \&c.\ \ldots\ \frac{(\mathrm{N}-1)\,s\,w'}{\mathrm{N}\,k};\ \ldots\ldots(11.)$$

the accuracy of the additions being tested by the direct computation of the last term of the series. Then compute the following series of N stresses, by beginning with 0, and adding successively the terms of the preceding series;

$$\left.\begin{array}{l} \mathrm{S}_0 = 0; \\ \mathrm{S}_1 = \frac{s\,w'}{\mathrm{N}\,k}; \\ \mathrm{S}_2 = \frac{s\,w'}{\mathrm{N}\,k} + 2\frac{s\,w'}{\mathrm{N}\,k}; \\ \&c. = \&c. \end{array}\right\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots(12.)$$

and test the accuracy of the calculation by the direct computation of the last term, viz. :—

$$\mathrm{S}_{\mathrm{N}-1} = \frac{(\mathrm{N}-1)\,s\,w'}{2\,k}\ \ldots\ldots\ldots\ldots\ldots\ldots\ldots(13.)$$

Divide this series of N terms into two halves, and range the terms of the second half beside those of the first half *in inverted order;* thus

$$\left.\begin{array}{ll} \mathrm{S}_0 & \mathrm{S}_{\mathrm{N}-1} \\ \mathrm{S}_1 & \mathrm{S}_{\mathrm{N}-2} \\ \mathrm{S}_2 & \mathrm{S}_{\mathrm{N}-3} \\ \cdot & \cdot \\ \cdot & \cdot \\ \cdot & \cdot \\ \mathrm{S}_n & \mathrm{S}_{\mathrm{N}-n-1} \\ \&c & \&c. \end{array}\right\} \ldots\ldots\ldots\ldots\ldots\ldots(14.)$$

Then for any given diagonal, whose number is n (that is, which lies between the joints n and $n+1$), the quantity corresponding to that number in the first column (S_n) will be the greatest stress produced by the rolling load, of the *contrary kind* (thrust or pull), to that produced by the fixed load; and the quantity in the same line of the second column (S_{N-n-1}) will be the greatest stress produced by the rolling load, of the *same kind* with that produced by the fixed load.

To find the greatest resultant Stress on each Diagonal Brace.

(1.) For the braces which slope *upwards* towards the middle of the span, take the sum of the stress due to the fixed load, and the greatest stress of the *same* kind due to the rolling load, as expressed by the formula,

$$T_n + S_{N-n-1}; \text{ .... (15.)}$$

the result will be the most severe stress, and will be a *thrust.*

(2.) For the braces which slope *downwards* towards the middle of the span, make the same calculation; the result will be the *greatest stress*, and will be *tension.*

But when a piece of wrought iron is exposed alternately to tension and thrust, the thrust, although less than the tension, may be *more severe*, on account of the smaller capacity of the material for resisting it. To ascertain whether this is the case for any particular brace sloping downwards towards the middle of the span, compare the *tension* produced by the fixed load (T_n) with the *greatest thrust* produced by the rolling load (S_n); and if the latter is the greater, the excess

$$S_n - T_n, \text{ .... (16.)}$$

will be the greatest thrust to be borne by the brace in question.

CASE II.—*The joints marked with even numbers loaded, the others unloaded.* In this case the full load is expressed as follows:—

$$W = (w + w') \cdot \left(\frac{N}{2} - 1\right). \text{ .... (17.)}$$

To find the Horizontal Stresses

compute the supporting pressure at the point 0 as follows:—

$$F_0 = \frac{W}{2} = (w + w') \cdot \left(\frac{N}{4} - \frac{1}{2}\right). \text{ .... (18.)}$$

Then compute the following series, of which the terms are equal by pairs, each pair being less than the preceding by the difference $\frac{l}{Nk}(w + w')$:—

$$\left.\begin{array}{l}\frac{l}{\mathrm{N}\,k}\,\mathrm{F}_1=\frac{l}{\mathrm{N}\,k}\,\mathrm{F}_0;\\ \frac{l}{\mathrm{N}\,k}\,\mathrm{F}_3=\frac{l}{\mathrm{N}\,k}\,\mathrm{F}_2=\frac{l}{\mathrm{N}\,k}\,\mathrm{F}_1-\frac{l}{\mathrm{N}\,k}\,(w+w');\\ \frac{l}{\mathrm{N}\,k}\,\mathrm{F}_5=\frac{l}{\mathrm{N}\,k}\,\mathrm{F}_4=\frac{l}{\mathrm{N}\,k}\,\mathrm{F}_3-\frac{l}{\mathrm{N}\,k}\,(w+w');\\ \quad\&\text{c.},\quad\&\text{c.},\quad\&\text{c.}\end{array}\right\}\ldots\ldots(19.)$$

The test of the accuracy of these calculations is, that for the division adjoining the middle joint, the result should be as follows:—

If the middle joint is loaded; that is, if $\frac{\mathrm{N}}{2}$ is even, $\frac{w+w'}{2}\cdot\frac{l}{\mathrm{N}\,k}$

If the middle joint is unloaded; that is, if $\frac{\mathrm{N}}{2}$ is odd; 0.

The series of horizontal stresses are computed by successive additions, precisely as in Case I., viz.:—

$$\mathrm{H}_1=\frac{l}{\mathrm{N}\,k}\,\mathrm{F}_0;\ \mathrm{H}_2=\mathrm{H}_1+\frac{l}{\mathrm{N}\,k}\,\mathrm{F}_1;\ \&\text{c.}\ \ldots\ldots\ldots(20.)$$

The test of the accuracy of this series of calculations is, that for the middle horizontal piece it should give a result agreeing with that of one or other of the following formulæ:—

If $\mathrm{N}\div 2$ is even,

$$\mathrm{H}_{\frac{\mathrm{N}}{2}}=\frac{\mathrm{N}\,l}{16\,k}\,(w+w');\ \ldots\ldots\ldots\ldots\ldots\ldots\ldots(21.)$$

If $\mathrm{N}\div 2$ is odd,

$$\mathrm{H}_{\frac{\mathrm{N}}{2}}=\frac{(\mathrm{N}^2-4)\,l}{16\,\mathrm{N}\,k}\,(w+w').\qquad .(22.)$$

To find the Diagonal Stresses,

the calculations are the same as in Case I., with the following modifications:—

Throughout all the calculations, $\frac{\mathrm{N}}{2}$ is to be substituted for N, that is to say, the girder is to be treated as having $\frac{\mathrm{N}}{2}$ instead of N divisions.

Each of the series (10), (12), has $\frac{\mathrm{N}}{2}$ instead of N terms.

The series (11) has $\frac{\mathrm{N}}{2}-1$ instead of $\mathrm{N}-1$ terms.

If $\mathrm{N}\div 2$ is odd, there will be a middle term in the series (12); and when the second half of the series is ranged in inverted order beside the first half, as in the table (14), that middle term is to be written at the bottom of each column.

Each of the results denoted by T and S in the series (10) and (14), and by their sums and differences in the formulæ 15 and 16, applies to a *pair* of adjacent diagonal braces, one sloping upwards and the other downwards towards the middle of the span. Thus

$$T_0,\ S_0,\ S_{\frac{N}{2}-1},\ \text{apply to the braces 0 and 1;}$$

$$T_1,\ S_1,\ S_{\frac{N}{2}-2},\ \text{,, ,, ,, 2 and 3;}$$

and generally,

$$T_n,\ S_n,\ S_{\frac{N}{2}-n-1},\ \text{apply to the braces } 2n \text{ and } 2n+1.$$

The ordinary angle of inclination of the braces in the Warren girder is 60°; in which case some labour of calculation is saved by the fact that the length of a brace, s, is equal to the distance from joint to joint along one of the booms, $2l \div N$.

EXAMPLE of Case II.—Suppose the design of the girder to be as in fig. 254, and to consist of 17 equilateral triangles, so that $N = 18$;

$$\frac{l}{Nk} = \frac{1}{\sqrt{3}} = \cdot 57735;\ \frac{s}{k} = 1 \cdot 1547;$$

also let the loads on each of the points 2, 4, 6, 8, 10, 12, 14, 16, be respectively

fixed, $w = 12{,}000$ lbs.

rolling, $w' = 18{,}000$ lbs.

This is nearly the case for a railway bridge girder of 160 feet span, supporting half the load of a line of rails. The supporting pressure is

$$F_0 = (w + w') \cdot \left(\frac{N}{4} - \frac{1}{2}\right) = 120{,}000 \text{ lbs.}$$

The following table shows the calculation of the horizontal stresses:—

$\frac{l}{Nk}(w+w')$	n	$\frac{l}{Nk}F$	H
Lbs.		Lbs.	Lbs.
	0,18	69282·0	
17320·5	1,17	69282·0	69282·0 thrust
	2,16	51961·5	138564·0 pull
17320·5	3,15	51961·5	190525·5 thrust
	4,14	34641·0	242487·0 pull
17320·5	5,13	34641·0	277128·0 thrust
	6,12	17320·5	311769·0 pull
17320·5	7,11	17320·5	329089·5 thrust
	8,10	0	346410·0 pull
	9	0	346410·0 thrust (Middle piece)

The verification of the last result is,

$$H_9 = \frac{(N^2 - 4)\,l}{16\,N\,k}(w + w') = 20 \times 17320{\cdot}5 = 346410.$$

The following are the calculations of the stresses on the braces, due to the fixed load:—

$$T_0 = \frac{s\,w}{k}\left(\frac{N}{4} - \frac{1}{2}\right) = 13856{\cdot}4 \times 4 = 55425{\cdot}6$$

$\frac{s\,w}{k}$	T	n, for braces to which the results are applicable as	
Lbs.	Lbs.	Thrust.	Pull.
	55425·6	1,16	0,17
13856·4			
	41569·2	3,14	2,15
13856·4			
	27712·8	5,12	4,13
13856·4			
	13856·4	7,10	6,11
13856·4			
	0	8,9	

The following table shows the calculation of the greatest stresses produced by the rolling load:—

$\frac{2\,s\,w}{N\,k}$		S	n, for braces to which the results are applicable as	
Lbs.	Lbs.	Lbs.	Thrust.	Pull.
		0	0,17	1,16
	2309·4			
2309·4		2309·4	2,15	3,14
	4618·8			
2309·4		6928·2	4,13	5,12
	6928·2			
2309·4		13856·4	6,11	7,10
	9237·6			
2309·4		23094·0	8,9	9,8
	11547·0			
2309·4		34641·0	10,7	11,6
	13856·4			
2309·4		48497·4	12,5	13,4
	16165·8			
2309·4		64663·2	14,3	15,2
	18475·2			
		83138·4	16,1	17,0

The verification of the accuracy of the additions is given by the following calculation:—

$$\frac{s\,w'}{k}\left(\frac{N}{4}-\frac{1}{2}\right)=20784{\cdot}6\times 4=83138{\cdot}4.$$

The following table shows the combined actions of the fixed and rolling loads on the braces, S′ denoting, for brevity's sake, the smaller value of S for the given brace. Thrusts are denoted by *t*, pulls by *p*:—

n	T	S	S′	T + S	S′ − T
	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.
0,17	55425·6 *p*	83138·4 *p*	0 *t*	138564·0 *p*	
1,16	55425·6 *t*	83138·4 *t*	0 *t*	138564·0 *t*	
2,15	41569·2 *p*	64663·2 *p*	2309·4 *t*	106232·4 *p*	
3,14	41569·2 *t*	64663·2 *t*	2309·4 *p*	106232·4 *t*	
4,13	27712·8 *p*	48497·4 *p*	6928·2 *t*	76210·2 *p*	
5,12	27712·8 *t*	48497·4 *t*	6928·2 *p*	76210·2 *t*	
6,11	13856·4 *p*	34641·0 *p*	13856·4 *t*	48497·4 *p*	0
7,10	13856·4 *t*	34641·0 *t*	13856·4 *p*	48497·4 *t*	0
8,9	0	23094·0 *p*	23094·0 *t*	23094·0 *p*	23094·0 *t*.

The accuracy of the numbers in the columns headed T + S and S′ − T may be checked by setting down the former in direct order, and the latter in inverted order, and taking their second differences, which ought to be constant, and equal to $2\,s\,w' \div N\,k$; that is, in the present case, 2309·4. The following is the process:—

	First Diff.	Second D
138564·0		
	32331·6	
106232·4		2309·4
	30022·2	
76210·2		2309·4
	27712·8	
48497·4		2309·4
	25403·4	
23094·0		2309·4
	23094·0	
0		

It appears from the values of S′ − T that in the example chosen the two middle braces alone act alternately as struts and ties under a rolling load.

It is unnecessary to give a numerical example of the calculations in Case I.; for they differ from those in Case II. only in being more simple.

IV. An *Iron Lattice Girder* consists essentially of a pair of

horizontal booms to resist the bending action of the load, and of two series of diagonal braces, inclined opposite ways, usually at 45°, to resist the shearing action. There may also be upright ribs, one at each loaded point, and one or more at each point of support, to distribute the load and the supporting pressures amongst the diagonal braces. Although these upright pieces are not absolutely essential, except at the points of support, it is advisable not to omit them. Their strength is to be fixed according to the same principles with that of the upright ribs of plate girders; Article 370, Division VI., p. 530.

To compute the stresses on the pieces of a lattice girder, one of its points of support is to be designated as 0 and the other as N, (N being the number of divisions into which the loaded points divide it); and the loaded points are to be numbered consecutively from 1 to N − 1.

In computations respecting the *shearing* action of the load, F_0 is to designate the shearing action in the division of the girder between 0 and 1, F_1 between 1 and 2, &c., and, generally, F_n between n and $n+1$; but in computations respecting the *horizontal stresses*, which depend on the bending action, H_1 is to denote the stress on the booms *at* a vertical section traversing the point 1, &c.

This being understood, the calculation of the thrusts and pulls on the horizontal booms is to be proceeded with precisely as for Case 1. of a zig-zag girder; formulæ 5 to 9, pp. 551, 552.

To find the stress on the lattice work, compute the two series of quantities $T_n + S_{N-n-1}$, $S_n - T_n$, for the several divisions of the girder, as for a zig-zag girder, Case 1., formulæ 10 to 16, pp. 552 to 554, and assume each of those forces to be equally distributed amongst the lattice bars that traverse the division of the girder to which it belongs. This assumption of equal distribution is not exact, but its errors are not of practical importance.

If the loaded points are numerous and near each other, the girder may be treated as an uniformly loaded beam, Article 161, Case VI., p. 246; and Case IX., p. 247.

V. *Zig-zag and Lattice Continuous Girders.*—In both these classes of girders the effect of continuity over the piers may be computed as follows:—

Calculate the horizontal stress on each division of the girder, when fully loaded, on the supposition that it is *discontinuous* at the piers; and let H_m be the result thus obtained for the middle horizontal bar. Then

$$H_p = \frac{2w + w'}{3w + 3w'} H_m; \quad \ldots\ldots\ldots\ldots\ldots \quad (23.)$$

will be the *tension on the upper boom*, and *thrust on the lower boom* of a continuous girder, over the piers, when its spans are alternately loaded and unloaded; and the *difference* between this and the stress H_u already calculated for any given bar of the girder supposed discontinuous, will be the stress on that bar when the girder is continuous and loaded on alternate spans; that is to say,

$$H_u - H_p. \text{ ....}(21.)$$

When this expression is negative (that is, when H_p is the greater term), the kind of stress is reversed. When it is $= 0$, it indicates a point of contrary flexure. (See p. 795.)

378. **Iron Braced Girders—Construction.**—I. *General Remarks.* Various iron trusses or braced girders have been made, in which the struts are of cast iron and the ties of wrought iron, advantage being thus taken of the greater resistance of cast iron to crushing and of wrought iron to tearing; but the greater liability to brittleness of cast iron, and the rapid diminution of its resistance to crushing as the proportion of length to diameter increases (as to which see Article 365, p. 520), have led to the general employment of wrought iron for the struts as well as for the ties, care being taken that the struts are of figures suited to resist a thrust, by having sufficient lateral stiffness. When a piece acts alternately as a strut and as a tie, it must have sufficient total sectional area, and sufficient stiffness, to resist the greatest thrust that can act, and sufficient effective sectional area to resist the greatest tension which can act along it. The straight lines of resistance which connect the centres of the joints with each other ought as nearly as practicable to coincide with the centres of the cross-sections of the several bars of the framing, in order to prevent unequal stress. (See paper by Mr. C. E. Reilly, *Proc. Inst.*, 25th April, 1865.)

II. The *Trapezoidal Truss*, already treated of in the preceding Article, p. 549, and represented in fig. 253, was used on an enormously large scale by the second Brunel in the railway viaduct over the Wye at Chepstow, the largest span of which, of about 300 feet, is crossed by two parallel and similar girders, of the following construction:—The horizontal strut B B B is a cylindrical plate iron tube 9 feet in diameter, and $\frac{5}{8}$ths of an inch thick, stiffened by transverse circular partitions or "diaphragms" at intervals. It is supported at the ends upon cast iron saddles, resting on cast iron pillars. The effective depth of the truss, denoted by k in the formulæ, is about 50 feet. Each of the principal ties, A, F, C, and of the diagonal braces, G, G, consists of a pair of flat-linked chains, attached to the sides of the tube, and sufficiently far apart at the level of the bottom of the truss to leave room for the

traffic on a line of rails between them. The vertical struts K, K, are rectangular frames with openings sufficient for the same traffic. The points 1 and 4 form the intermediate supports of a pair of ordinary plate iron girders, whose ends rest on the piers; so that for those girders the span of 305 feet is subdivided by the aid of the truss into three spans; the central span being 124 feet, and the side spans each $90\frac{1}{2}$ feet. Below the four plate girders, thus hung from the two great trusses, are attached the cross joists of the roadway. The two tubes are braced together horizontally to increase their lateral stiffness.

The total fixed load of this structure *for one line of rails* is about $1\frac{1}{2}$ tons per lineal foot, or 3,360 lbs. The greatest rolling load may be taken, as was usual in railway bridges, at one ton per foot, or 2,240 lbs.

The following are the proportions per cent. in which the fixed load is distributed:—

Effective section of tube, ...	20	per cent.
Other parts and appendages of tube,...	15	,,
Main chains, ...	23	,,
Diagonal chains, ...	5	,,
Upright frames, saddles, &c., ...	9	,,
Plate girders, joists, and roadway,.. ...	28	,,
	100	,,

With the fixed and rolling loads above-mentioned, the thrust along the tube is about 3,600 lbs. per square inch of section; so that the factor of safety is considerably more than six for both fixed and rolling loads.

III. *Warren or Zig-zag Girders.*—In the earlier examples of these girders, the upper horizontal strut or boom was a tube, cylindrical inside, and on the outside resembling a cylinder with four flat projections above, below, and at each side, for the convenience of attaching the diagonals to it. The strut braces were of cast iron, and cross-shaped. In later examples the upper boom and strut braces are made of wrought iron; the upper boom being either like a trough-shaped girder built of flat bars and angle iron, with the flanges downwards, or like a box-beam, and the strut-braces H-shaped, or cross-shaped, as shown in fig. 235, p. 521. The main tie or lower boom, and the tie-braces, consist of flat links set on edge; as to which, see Article 364, Divisions II. and III., pp. 518, 519. The joints of the lower boom, and its connections with the braces, are made by means of large cylindrical pins. Such pins also connect the braces with the upper boom, whose side plates or bars form a channel into which the ends of the braces enter and

fit. The joints of the horizontal tie may also be made by fishing and rivetting. The whole structure has the advantage of being easily carried in pieces to its intended site, and there put together.

If the platform is hung below the girders, lateral stability is to be given to them by making the vertical suspending-pieces, whereby the cross joists are hung from the higher joints of the girders, of an I shaped form of section, and equal, or nearly equal in stiffness, to the platform joists. When the platform is supported above the girder, lateral stiffness is to be given by the horizontal diagonal bracing of the platform, and also by vertical transverse diagonal bracing between the girders; and for this purpose rods of from 1 inch to $1\frac{1}{2}$ inch in diameter are in general sufficient. (For details of various Warren girders, see Humber *On Iron Bridges.*)

From the manner in which the parts of a zig-zag girder are connected together, it is evident that its diagonal strut-braces, and the several divisions of its horizontal boom, are to be treated as *struts hinged at the ends.* (See Article 366, p. 523.)

IV. *Lattice Girders.*—The forms and modes of construction applicable to the upper and lower booms of the Warren girder are also applicable to those of the lattice girder. The diagonal strut-braces are made of any convenient shape that is well suited to resist thrust; their greatest breadth should be placed *transversely,* because in the longitudinal plane of the girder, they are stiffened by being bolted or rivetted to the tie-braces at each intersection. The holes made for that purpose weaken the tie-braces, and are to be allowed for in computing their strength. In the Boyne Viaduct, the strut diagonals of the lattice girders are themselves formed like small lattice beams, consisting of a pair of T-iron ribs connected together by small diagonal braces. (See p. 800.)

379. **Iron Bowstring Girders.**—The most common kind of iron bowstring girder (fig. 256) consists of a cast or wrought iron arch or bow, springing from two shoes or sockets, which are tied

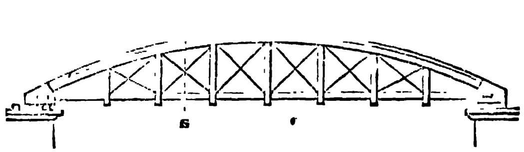


Fig. 256.

together by a horizontal tie; the cross joists of the platform are suspended from the arch by vertical suspending-pieces, which at the same time support the weight of the tie; and stiffness to resist a rolling load is given by means of diagonal tie-braces.

The proper figure for the centre line or neutral curve of the bow

is a parabola; but a circular segment is often used in practice. The cross-section of the bow, like that of the upper boom of a lattice girder, must be of a form suited for resisting thrust. A cylindrical tube is the strongest form; an inverted trough-shape, either cast, or built of plates and angle bars, is convenient for the attachment of the suspending-pieces. These have usually an I-shaped section, with the greatest breadth transverse, to give them lateral stability; and for the same purpose they widen towards the bottom, where they are rivetted to the ends of the plate or box beams that form the cross joists. The main tie is best made of parallel flat bars on edge, and is made fast to the shoes at each end by gibs and cotters; the diagonal braces are round or flat rods. The stress on each part is found as follows:—

CASE I.—The girder stiffened by diagonal braces.

Let l be the span, measured along the centre line of the main tie, between the ends of the centre line of the bow;
k, the rise from centre line to centre line;
w, the fixed load, and w', the rolling load, } per unit of length of span.

Then the tension of the main tie, and the horizontal thrust at the crown of the bow, are given very nearly by the formula

$$H = (w + w')\, l^2 \div 8\, k. \;\ldots\ldots\ldots\ldots\ldots\ldots (1.)$$

The thrust at any other point of the arch varies nearly as the secant of its inclination; or, to express it in symbols, let x be the horizontal distance of the point in question from the middle of the span; then the thrust is,

$$\sqrt{\{H^2 + (w + w')^2\, x^2\}}\;\ldots\ldots\ldots\ldots\ldots\ldots (2.)$$

At the springing, for x put $l \div 2$.

Let N be the number of parts into which the vertical pieces divide the span, so that there are N − 1 of those pieces; the greatest tension on any one of them is nearly

$$\frac{(w'' + w')\, l}{N};\;\ldots\ldots\ldots\ldots\ldots\ldots (3.)$$

w'' being the fixed or dead load per unit of length, exclusive of the weight of the bow.

It is possible that when the girder is partially loaded with a travelling load, some of the upright pieces, which, with a uniform load, act as ties, may be made to act as struts. To find whether this is the case, number the uprights from one end of the girder;

let n be the number of any given upright; compute the value of the expression

$$\frac{l}{\text{N}}\left(w' \cdot \frac{n\,(n+1)}{2\,\text{N}} - w''\right); \ldots\ldots\ldots\ldots\ldots\ldots(4.)$$

if this is positive, it gives the greatest thrust on the upright in question; or negative, it shows that the upright never acts as a strut.

The greatest tension produced by a rolling load on any given diagonal brace is given by the expression,

$$\frac{w'\,l\,s}{y} \cdot \frac{n\,(n+1)}{2\,\text{N}^2}; \ldots\ldots\ldots\ldots\ldots\ldots(5.)$$

where s is the length of the given brace, y the difference of level of its ends, and n and $n+1$ the numbers of the uprights between which it is placed.

Case II.—The girder without diagonal braces. In this case the action of a rolling load must be resisted by the stiffness of the bow. The greatest stresses on the tie and on the suspending-pieces are given by the expressions (1) and (3) respectively; but the bow becomes virtually *an arched rib hinged at the ends*, as to which see Article 374, Case III., p. 540, and Article 180, Problem VI., pp. 310 to 312. In computing the quantity C by equation 51 of the last-mentioned Article, p. 310, the effect of change of temperature is *not* to be considered, because the bow and tie expand equally; and the term denoted by $a\,\text{E}\,\text{A}_1 \div l$ in that equation is to be replaced by $u_a \div u_b$, denoting the ratio which the greatest safe shortening of the bow bears to the greatest safe lengthening of the tie. If they are both of wrought iron, this may be assumed as approximately $= \frac{1}{2}$.

Case III.—*Bowstring Suspension Bridge.*—(Fig. 257.) In this class of bridge, of which the greatest example is that erected by the

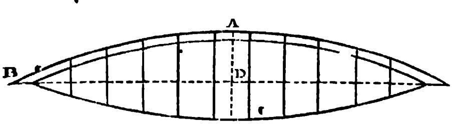


Fig. 257.

second Brunel over the Tamar at Saltash, the tie hangs in a catenary curve, and assists the bow in supporting the vertical pieces. The bow is a wrought iron oval tube stiffened by transverse diaphragms; the tie consists of a pair of chains.

In fixing the proportions of a bridge of this kind, it is advisable

to make the horizontal thrust due to the weight of the bow balance the horizontal tension due to the weight of the tie, independently of any additional load; and for that purpose, the common horizontal chord of the two arcs should divide the greatest vertical distance between their lines of resistance, A C, at the point D, into segments proportional to their weights; that is to say,

$$\begin{array}{ccccc} \text{Weight of bow} & : & \text{weight of tie} & : & \text{sum of weights} \\ \text{A D} & & \text{D C} & & \text{A C} \ k. \end{array}$$

The formulæ applicable to this case are the same with those for Case I. or Case II., according as the girder is enabled to resist a travelling load by means of diagonal braces, or by the stiffness of the bow alone.

380. **Braced Iron Arches.**—This term is applied to arches in which the arched rib and horizontal rib are so connected together by zig-zag braces (as in fig. 258), or by lattice work, in the intervening spandril, that each half-arch, together with its spandril, forms one stiff frame or truss. The best examples of this kind of arch are made of wrought iron; amongst them may be mentioned, the railway bridge over the Theiss at Szegedin, by M. Cezanne (*Annales des Ponts et Chaussées*, 1859), which consists of eight arches of 41·118 mètres in span (135·88 feet), and the bridge of the Paris and Creil railway over the Canal Saint-Denis, by M. M. Salle and Mantion (*Annales des Ponts et Chaussées*, 1861), consisting of a single arch of dimensions which will presently be stated. Each of those structures consists of four parallel frames, one under each rail; each frame consists of a curved rib, a straight horizontal rib, and a zig-zag series of braces, alternately vertical and sloping; these pieces are built of plate and bar iron, rivetted together so that the whole frame acts like one piece.* In the Theiss bridge, all the pieces are I-shaped in section, consisting of a middle web with flanges or tables connected to it by angle irons and rivets; in the Paris and Creil railway bridge, the horizontal rib is I-shaped, the braces are cross-shaped, and the curved rib consists of a vertical web, with four Barlow rails rivetted to it, two on each side; and for about one-eighth of the span on each side of the crown, the horizontal rib and curved rib have no opening between them, so that the same vertical web serves for both. The four parallel frames of the arch are stiffened transversely by two sets of T-shaped

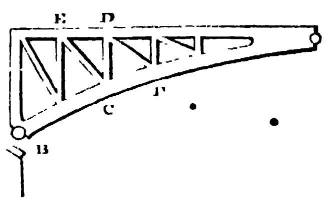


Fig. 258

diagonal stays, one connecting the horizontal ribs together, and the other the curved ribs.

It has already been stated in Article 374, p. 540, that M. Mantion proposes, as the best mode of constructing an iron arch, to have hinges at the crown and at the springing, as at A and B, fig. 258. The arches of the Paris and Creil railway bridge are hinged at the springing, but continuous at the crown; those of the Theiss bridge are continuous at the crown, and have flat abutting surfaces at the springing; nevertheless, from the smallness of those surfaces as compared with the other dimensions of the arch, it is probable that the arches of this bridge also act nearly as if they were hinged at the springing.

The following are some of the principal dimensions of the Paris and Creil railway bridge:—

The length of each semi-arch is divided into ten equal divisions horizontally; there are in each spandril eight vertical and six diagonal braces; for two divisions and a-half adjoining the crown there are no braces, the curved and straight ribs having one web in common.

		Mètres.	Feet.
Span between axis of bearings,		44·846	147·14
Rise,		4·85	15·91
			Inches.
Depth of curved rib (= span ÷ 66 nearly),		0·680	26·77
" of straight rib,		0·300	11·81
" of combined rib at crown,		0·705	27·76
" of braces, four longest at each end,		0·200	7·87
" of braces, remainder,		0·150	5·91
Breadth,	of T-shaped transverse braces of curved ribs,	0·150	5·91
Depth,		0·080	3·15
Breadth,	of T-shaped transverse braces of straight ribs,	0·125	4·92
Depth,		0·060	2·36
Length,	of semi-cylindrical bearings for ends of curved ribs,	0·302	11·89
Diameter,		0·200	7·87
Length,	of cast iron abutting-plate, which carries semicylindrical bearing,	1·40	53·11
Breadth,		1·00	39·37
		about	about
Mean thickness,		·080	3·15

Areas of Cross-section.		Square Millimetres.	Square Inches.
Combined rib at crown,		59,400	92·07
Curved rib in five divisions adjoining the crown,	from	77,650	120·36
	to	42,050	75·18
Curved rib—Remainder,		35,950	55·72
Horizontal rib	from	23,200	35·96
	to	9,700	15·04
Braces	from	7,482	11·60
	to	11,505	17·83
Transverse diagonal stays,	from	2,760	4·28
	to	1,665	2·58

The horizontal ribs perform the duty of beams in supporting cross joists; for besides the joists that lie directly above the uprights of the spandril, there are two intermediate joists at equal distances resting on the horizontal ribs in each space between a pair of uprights.

The following is the load of the structure:—

	Total for Double Line. Kilogrammes.		Lbs.		Lbs. per Lineal Foot of Single Line.	
Wrought iron framework,.........		125,000		275,578		936
Dead load exclusive of iron frame, viz :—						
Timber platform, ...	45,000		99,2[illegible]		337	
Ballast,..............................	45,125		99,483		338	
Rails,................................	6,875	97,000	15,157	213,848	52	727
Total dead load, ...		222,000		489,426		1,663
Greatest working live load, estimated at 4,000 Kilogrammes per mètre of single line, on 90 mètres,............		360,000		793,663		2,697
Total greatest working load,......		582,000		1,283,089		4,360

The following are the methods of computing the stresses on the several pieces of a braced iron arch:—

For the uprights and sloping braces, use the same rules as for the suspending-rods and diagonal braces of a bowstring girder. (See Article 379, p. 563). For the arch proceed as follows:—

CASE I.—*When the arch is hinged both at crown and springing,* the most severe stress on the arc and on the horizontal rib are determined as follows, with an approximation sufficient for practical purposes (see fig. 259):—

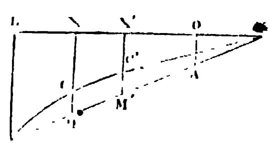


Fig. 259

Let w be the dead load per lineal foot;

w', the live load per lineal foot;

c, the *half* span, } of the centre line of the arched rib in

k, the rise, } feet.

Then the horizontal thrust due to the dead load is,

$$\mathrm{H}_0 = \frac{w\,c^2}{2\,k}; \text{ ....(1.)}$$

and to a live load over the whole span,

$$H_1 = \frac{w' c^2}{2 k}; \qquad (2.)$$

and if this be the most severe way of loading the arch, the required sectional area at any given point of the arched rib, where its inclination is i, will be,

$$A = \frac{(H_0 + H_1) \sec i}{f'}; \qquad (3.)$$

f' being the *safe working* thrust on the material; or say about 6,000 lbs. per square inch.

To ascertain the effect upon the curved and straight ribs, of loading one-half of the arch with the live load, and leaving the rest unloaded, either a geometrical or an algebraical method may be followed. For the geometrical method, let A B, fig. 259, be the centre line of the curved rib, O L that of the straight rib; join A B with a straight chord. Let X C M be any vertical ordinate. Then the stress along the horizontal rib at X is,

$$\frac{H_1 \cdot M C}{2 C X}; \qquad (4.)$$

and this is tension when X is in the unloaded half of the span, and thrust when it is in the loaded half.

The horizontal component of the greatest stress arising from a rolling load on half the span, at the point C in the arched rib, occurs when C is in the unloaded half of the rib, and is as follows:—

$$\frac{H_1 \cdot M X}{2 C X}; \qquad (5.)$$

and should this prove *greater* than H_1, that is to say, should M X be greater than 2 C X, the expression (5) is to be substituted for H_1 in equation 3; but should M X be not greater than 2 C X, equation 3 is to be left unaltered.

To find the point of *greatest* horizontal stress in the unloaded half of the beam, produce the straight lines L O, B A, till they meet in N, from which draw N C′ touching the curve A C B; C′ will be the point sought.

The algebraical formulæ for the expressions (4) and (5) are as follows:—

Let O A $= a$; O X $= x$; then,

$$\frac{H_1 \cdot M C}{2 C X} = \frac{H_1 (k c x - k x^2)}{2 (a c^2 + k x^2)}; \qquad (4 \text{ A.})$$

$$\frac{H_1 \cdot M X}{2 C X} = \frac{H_1 (a c^2 + k c x)}{2 (a c^2 + k x^2)} \dots\dots\dots\dots (5 \text{ A.})$$

The value of x which makes the last expression a maximum is given by the equation

$$O X' = x' = c \left\{ \sqrt{\frac{a}{k} + \frac{a^2}{k^2}} - \frac{a}{k} \right\} \dots\dots\dots\dots (6.)$$

It is to be observed that the processes expressed by the formulæ 4, 5, 4 A, 5 A, 6, are applicable only to the *open-work* parts of the frame. Where the horizontal rib and the arched rib are connected by a web, so as to form virtually *one rib*, that rib is to be conceived to be under the combined action of the thrust $H_0 + \frac{H_1}{2}$, and the bending moment

$$M' = \frac{H_1 \cdot M C}{2} = \frac{H_1 k (c x - x^2)}{2 c^2} \dots\dots\dots\dots (7.)$$

Let h be the depth of the compound rib, and q a co-efficient depending on its form of section, as given in pp. 294, 295. Then its sectional area is given by the equation

$$\left. \begin{aligned} A' = \frac{1}{f'} \left(\frac{M}{q h} + \frac{H_1}{2} + H_0 \right) = \frac{H_1}{2 f'} \left(\frac{k (c x - x^2)}{q h c^2} + 1 \right) \\ + \frac{H_0}{f'}; \end{aligned} \right\} (8.)$$

and *if this area is greater than that given by equation* 3, it is to be adopted.

Case II.—*When the rib is continuous at the crown*, the *exact* determination of the state of stress at different points becomes a problem of almost impracticable complexity; but an approximate solution, sufficient to determine what sectional area is required at and near the crown, in order to resist the straining effects of deflection, yielding of the piers, and changes of temperature, may be obtained as follows:—

Compute a series of values of the expression $q m' h^2$, as explained in Article 180, p. 302, equation 17, for a series of equidistant cross-sections of the entire iron frame, and use the *mean* of all those values to compute the quantity C by the following formula:—

$$C = \frac{15 q m' h^2}{8 k^2} \left(1 + \frac{4 k^2}{3 c^2} + \frac{2 a E A_1}{c} \mp \frac{e t E}{p_0} \right); \dots (9.)$$

in which a is the enlargement of span due to yielding of the piers

per lb. of thrust; A_1, an assumed approximate sectional area of the curved rib; E, the modulus of elasticity; $\mp t$, the extreme $\left\{\begin{matrix}\text{rise}\\\text{fall}\end{matrix}\right\}$ of temperature; e, the co-efficient of expansion per degree (see p. 539), p_0, the intended mean intensity of thrust at the crown.

Then calculate a moment of flexure as follows:—

$$M'' = \frac{(w + w')c^2}{2} \cdot \frac{C}{1 + C} = (H_0 + H_1)\frac{C k}{1 + C}; \quad (10.)$$

let h_0 be the depth of the rib *at the crown*, and q_0 the value of q for the same point; then the corrected sectional area at the crown will be,

$$\left.\begin{aligned} A = \frac{1}{f'}\left(\frac{M''}{q_0 h_0} + H_0 + H_1\right) = \frac{H_0 + H_1}{f'} \\ \left(\frac{C k}{(1 + C) q_0 h_0} + 1\right). \end{aligned}\right\} \quad ...(11.)$$

When the horizontal rib of a braced iron arch acts also as a beam, the sectional area required to resist at once the direct stress and the bending action is to be computed according to the principle of Article 374, Case III., equation 2, p. 540.

381. **Iron Piers.**—An iron pier for supporting arches or girders may consist of any convenient number of hollow cylindrical pillars, either vertical or raking, each pillar being made of pieces of a convenient length, turned or planed at the ends, and united by a projection and socket, and also by flanges or lugs and bolts, as explained in Article 365, p. 521, and the several pillars being connected together by horizontal and diagonal braces. For the method of determining the stress on each pillar and brace, see Article 318, pp. 484, 485. Each *length* of a pillar between a pair of braced points may be considered as a strut *hinged at the ends*, and its strength computed accordingly. (See Article 365, p. 521; also pp. 795, 800.)

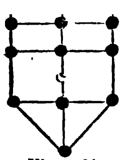


Fig. 260.

As an example of piers constructed in this manner, may be taken those of the Crumlin Viaduct (fig. 228, p. 491), in which the greatest height of the rails above the valley is about 200 feet. Each pier consists of fourteen cast iron columns, in lengths of 17 feet, with an uniform external diameter of 12 inches, and a thickness of metal ranging from one inch at the base to $\frac{7}{8}$ inch at the top. The two centre columns are vertical; the remainder rake in such a manner that while the base of the highest pier measures 60 feet by 27, the top of each pier measures 30 by 18. The

longitudinal and transverse horizontal braces are cast iron beams, I-shaped in section, and 12 inches deep; their flanges are 5 inches broad. The diagonal braces in vertical and raking planes are flat bars measuring 4 inches by $\frac{3}{4}$ inch; there are also horizontal diagonal braces, which are round rods of 2 inches diameter. Each column has a foot or base from 3 feet to 5 feet high, spreading to 3 feet square, and resting on a foundation of masonry to which it is bolted and joggled. (See authorities, p. 531.)

Wrought iron struts of suitable figures may be used instead of cast iron pillars in the construction of piers; and like them, they are to be considered as hinged at the points which are fixed by the braces. (See Article 566, p. 521.)

In some cases a pier is made of a single row of hollow cylindrical cast iron pillars, or even of a single such pillar; in which case the greatest intensity of tension and of thrust are to be computed as follows:—Let P be the vertical load of one pillar; H, the horizontal thrust applied to it, at a height of Y above its base, or above the horizontal section at which the stress is to be calculated; d, the mean between the external and internal diameters of the pillar; A, its sectional area ($= 3{\cdot}1416\, d \times$ thickness of metal); then

$$\text{greatest intensity of} \left\{ \begin{array}{l} \text{thrust} \\ \text{tension} \end{array} \right\} = \frac{1}{A} \left(\frac{4\,H\,Y}{d} \pm P \right) \text{ nearly. (1.)}$$

Cases in which the bending moment arising from the thrust differs from H Y will be considered further on.

When a pillar simply rests on a firm base, without being imbedded in the soil like a pile, it is advisable so to proportion it that there shall be no tension at any point of its base; and for that purpose the diameter at the base should not be less than that given by the following formula:—

$$d = \frac{4\,H\,Y}{P} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.)$$

As examples of piers of this class, may be taken those used for the bridges of the Bombay and Baroda railway, by Lieutenant-Colonel Kennedy (see *Civil Engineer and Architects' Journal*, September, 1861), each consisting of three hollow cylindrical vertical cast iron pillars, connected together by horizontal and diagonal braces, with the addition, in powerful currents, of a pair of raking struts of the same dimensions and construction with the pillars, making angles of 30° with the vertical. The pillars are cast in lengths of 9 feet, and are 2 feet 6 inches in external diameter, and 1 inch thick; the lengths are connected together by flanges and bolts. For the part above ground the flanges are

external, and have each 12 bolts of 1 inch diameter; for the part below ground, they are internal, and have each 10 bolts; and the diameter above-mentioned has been adopted as the least which will easily admit of a workman's going inside to fasten the bolts of the internal flanges. In foundations in earth the lowest length forms a screw-pile, with a screw 4 feet 6 inches in diameter, by means of which the pillar is screwed from 20 to 45 feet into the ground according to the softness of the material. Further mention of such piles will be made in a subsequent chapter, under the head of "Timber and Iron Foundations." When the ground consists of rock, each pillar is inserted into a cylindrical hole about 2 feet deep, and fixed there with cement. The three vertical pillars stand at distances of 14 feet from centre to centre. The horizontal braces are of T-iron, of a sectional area between 5 and 6 square inches; the diagonal braces are of angle iron, of a sectional area between 3 and 4 inches; each brace is fastened to lugs on the pillars, and tightened at one end by a gib and cotter.

The piers just described have lateral stiffness sufficient to withstand a current, if free from floating ice and large trees; but they are not adapted to bear the thrust of an arch, unless it be one of very small size. The superstructure of the bridges in which they are used consists of Warren girders.

In some lately erected bridges, the cast iron columns which form the piers are cylinders of 7 feet, 9 feet, 10 feet, and upwards, in diameter, and from 1 to 2 inches thick, filled with concrete or with rubble masonry. The mode of sinking such cylinders will be described under the head of "Timber and Iron Foundations." They are capable of withstanding a considerable thrust from an arch.

For example, in the Theiss bridge, mentioned in Article 380, p. 565, each pier consists of two cylinders, side by side:—

The diameter of each cylinder is 3 mètres, or	9·843 feet.
The thickness,.......................... about	1·38 inch.
The depth of the springing of the arches below the centre line of the horizontal ribs,....................................	$Y' = 18·93$ feet.
The height of the springing of the arches above the base of the pier,..............	$Y = 65·4$ feet.
The greatest thrust against a column occurs when one of the arches springing from it is fully loaded, and the other unloaded; in this case the vertical load *on one column* is,..........................	$P = 368{,}000$ lbs.
And the excess of the thrust of the loaded over that of the unloaded arch,	$H = 361{,}500$ lbs.

M. Cezanne, in his account of this bridge before referred to, considers the column as a *vertical beam*, acted upon by the pressure H at the springing of the arch, which is resisted by the thrust of the *horizontal rib of the unloaded arch* at the top of the column, and by that of the foundation at its base, so that the bending moment, instead of being H Y, as in equation 1 of this Article, is

$$\frac{H\,Y\,Y'}{Y + Y'};\ \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (3.)$$

and the greatest

$$\left.\begin{array}{l}\text{thrust}\\ \text{tension}\end{array}\right\} = \frac{1}{A}\left(\frac{4\,H\,Y\,Y'}{(Y + Y')\,d} \pm P\right).\ \dots\dots\dots\dots (4.)$$

According to these principles, the greatest intensities of the stress in the cylinders of the Theiss bridge are,

Thrust, about 4,300 lbs. per square inch.
Tension, about 730 „ „

382. **Suspension Bridges.**—I. *Figure, Weight, Arrangement, and Loading of Chains or Cables.*—The whole theory of the action of an uniformly distributed load on a suspension bridge, when the suspending-rods are vertical, has been given in Article 125, pp. 188 to 191, and when the suspending-rods are oblique, in Article 126, pp. 191 to 194.

It is advisable to make the factor of safety for the fixed load *three*, and that for the rolling load *six*; but in many actual suspension bridges the factors are much less.

When, for reasons of practical convenience, each chain is made of uniform sectional area, that area must be proportioned to the greatest pull; that is to say, to the pull at the points of support (or at the highest point of support, if their heights are unequal); but a saving may be made, both of load and of material, by making the sectional area of the chain at different points vary as the pull; that is to say, as the secant of the angle of inclination of the chain. The weights of sections of the chain, extending over *equal horizontal distances*, will in this case vary as the *squares* of the secants of their angles of declivity.

The following formulæ show both the absolute and comparative weights of chains of uniform section and of uniform strength, to a degree of approximation sufficient for practical purposes:—

Let x be the half-span of the chain; y, its depression, both in feet; the ordinary proportions of x to y range from $4\frac{1}{2} : 1$ to $7\frac{1}{2} : 1$.

Let C be the weight of a chain of the length x, and of a

cross-section sufficient to bear safely the greatest working horizontal tension H.

C′, the weight of a *half-span* of the chain of *uniform section*.

C″, the weight of a *half-span* of the chain of *uniform strength*; then,

$$C' = C \cdot \left(1 + \frac{8}{3}\frac{y^2}{x^2}\right) \text{ nearly. } \ldots\ldots\ldots\ldots\ldots (1.)$$

$$C'' = C \cdot \left(1 + \frac{4}{3}\frac{y^2}{x^2}\right). \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.)$$

The error of the first formula is in excess, and does not exceed 1-3000th part in any case of common occurrence in practice.

The value of C in the above formulæ may be taken as follows:—

$$\text{For wire cables of the best kind, } C = \frac{H\,x}{4500}; \ldots\ldots (3.)$$

$$\text{For cable-iron links, } C = \frac{H\,x}{3000}; \ldots\ldots\ldots\ldots (4.)$$

it being understood that the last formula gives the *net* weight only; in other words, the weight exclusive of the additional material in the eyes and pins by which the links are connected together.

About *one-eighth* may be added to the net weight of the chains, for eyes and fastenings.*

As to the structure and mode of connection of flat-linked chains and wire cables, see Article 361, Divisions III., V., pp. 519, 520.

The smallest number of chains or cables in a suspension bridge is two, one to support each side of the roadway. In other cases there are from two to four parallel sets of chains, each consisting of two or more chains in the same vertical plane. For example, in the Menai Bridge, there are sixteen chains, in four sets of four.

The equal distribution of the load amongst a set of chains which hang in one vertical plane may be effected in different ways; one being to distribute the suspension-rods equally amongst them. In order that this plan may be effective, all the chains should be of

* A great improvement in the manufacture of bars for bridge chains, introduced by Messrs. Howard and Ravenhill, consists in a process of rolling them with enlarged heads on their ends, so that the eyes can be made without forging or welding.

Here may be mentioned the test applied by Mr. Page to the bars used for the chains of Chelsea Suspension Bridge (which test has been omitted from its proper place in Article 357). Each bar was subjected to a tension of the intensity of 13½ tons (or 30,240 lbs.) per square inch; and if, after the removal of the stress, the length of the bar was found to be *permanently* increased by more than 1-400th inch per foot (or 1-4800th), it was rejected. The ultimate tenacity of bars which withstood this test was found to be 31 tons, or 69,440 lbs. per square inch.

exactly equal span, depression, and dimensions, so that they may all be affected alike by changes of temperature and of load.

The method which insures the most accurately equal distribution of the load on two chains, is that used by Brunel in the late Hungerford Bridge, and represented in fig. 261; A is a suspension-rod, hanging from the middle of a small wrought iron lever, B, of equal arms; the ends of that lever are hung by rods C, D, from the two chains E, F, each of which bears exactly half the load of the rod A.

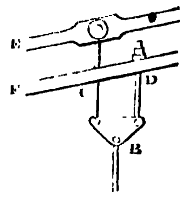


Fig. 261.

In Chelsea Bridge the rods C and D are dispensed with; and the lever B becomes a sort of scalene triangle, whose two upper angles are supported, one on the joint pins of one chain, the other by a pin resting on the top of the other chain, while from its lowest angle hangs the rod A.

Each suspending-rod should have its length capable of adjustment, by means of a screw, arranged according to convenience.

The ordinary distance between the suspending-rods is from 5 to 12 feet; and each of them carries one end of a cross joist of the platform.

When a suspension bridge consists of several bays or spans, the chains of all of them must form portions of equal and similar parabolas (the parabola being considered a sufficiently close approximation to the true curve in which the chain hangs, as already explained in Article 128, pp. 197, 198)

II. *Platform.*—On this point see what has already been stated as to timber platforms in Article 336, pp. 465 to 468, and iron platforms, in Article 375, pp. 542 to 546.

The platform of a suspension bridge is usually cambered, or slightly arched upwards.

III. *Piers and Saddles.*—As to the properties of different methods of supporting the chains on the tops of the piers, see Article 125, Problem VI., p. 191. Unless the pier has considerable stability, the second construction there described, viz.:—That in which the chains are made fast by pins to a truck, supported on rollers on a level base or platform, is to be preferred, as insuring that the load on the pier shall be exactly vertical. From good practical examples it appears that the length of the platform on which the rollers rest may be about one-fiftieth part of the span of one bay of the chains. The truck should be of wrought iron.

In some cases *hinged cast iron piers* have been used, each of which has the chains made fast to its upper end, while, at its lower end, it is capable of turning through a small angle in a vertical

plane, about a horizontal axis, so as to lean slightly inwards or outwards as the distribution of the load varies.

Mr. P. W. Barlow has proposed, in order to diminish or prevent the disfigurement of a suspension bridge of many bays, when one bay is loaded and the adjoining bays unloaded, that the ends of the chains of each bay should be made fast to the top of a wrought iron pier, constructed like a plate girder set on end, and having strength and stability sufficient to resist the excess of horizontal tension in the loaded bay above that in the unloaded bay.

Let w' denote the greatest *travelling* load per foot of span;

x, the half-span of a bay;

y, the depression of each chain;

then the excess of horizontal tension in question is

$$H' = \frac{\quad}{2\,y}. \qquad (5.)$$

and this being multiplied by the depth of any given horizontal section of the pier below the point of attachment of the chains, gives the bending moment at that section. The vertical stress produced by that moment, compressive at one side of the pier and tensile at the other, is combined with the compressive vertical stress produced by the total load, whose amount is as follows:—

Let w be the fixed load, per foot of span;

W'', the weight of the pier itself, above the given horizontal section; then the load is

$$P = W'' + (2\,w + w')\,x. \,\ldots\ldots\ldots\ldots\ldots\ldots\,(6.)$$

As to the combined action of the load and bending moment, see Article 381, p. 571.

IV. *Abutments—Anchoring Chains.*—The term "Abutment" is applied to those masses, whether of masonry or of natural rock, to which the extreme ends of the chains are made fast, and by whose stability the tension of the chains is resisted. For example, in fig. 262 (which bears a general likeness to an abutment of the late Hungerford Bridge), a pair of chains enter an opening in the abutment at A, in a direction nearly horizontal. At B their direction is changed to one more steeply inclined, by the aid of a saddle, which presses against the masonry in front of it. The

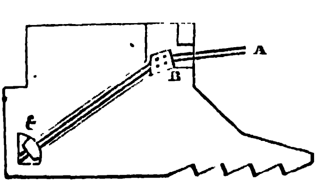


Fig. 262.

chains traverse a sloping tunnel or passage in the abutment, and finally pass through holes in the "anchoring plates" of cast iron at C, where they are fixed by keys or wedges; the anchoring plates press against a pair of transverse cast iron girders imbedded in the masonry.

Except that the tendency is to upset or to slide forwards instead of backwards, the principles of the stability of the abutment of a suspension bridge are precisely the same with those of the abutment of an arch; that is to say, the weight of the abutment must be sufficient to prevent it by friction from sliding on its base; its weight and thickness must be sufficient to prevent it from upsetting; and the centre of resistance of its base must not deviate from the centre of figure by more than a safe fraction of the thickness. As to ordinary foundations for such abutments, see Articles 235 to 239, pp. 377 to 382; as to the stability of the abutments, see Articles 263, 264, pp. 396 to 401. The resistance to sliding forward may be increased by making the base of the abutment, or part of it, slope so as to be perpendicular, or nearly so, to the resultant pressure, as in the front part of the abutment in fig. 262.

When piles are used in the foundation, they should be driven as nearly as possible in the direction of the resultant pressure. (See Section II. of the next chapter.)

The saddles by the aid of which the direction of a chain within its abutments is changed, do not require rollers, though they must be capable of sliding to an extent sufficient to admit of the expansion and contraction of the chain. This has been effected by making them rest on a bed about 4 or 5 inches thick, consisting of layers of asphalted felt.

As wire cables, from their great extent of surface, require more care in order to prevent them from rusting than bars, it is generally considered advisable that chains made of bars should always be used *within the abutments* of suspension bridges, although to the outer ends of such chains wire cables of equal strength may be attached. The cavities and passages containing these anchoring chains and their fastenings ought to be accessible for purposes of examination, painting, and repair.

V. *Oscillations and Means of Checking them.*—A suspension bridge consisting simply of abutments, piers, chains, vertical suspending-rods, and load, is free to oscillate both vertically and horizontally, the vertical oscillations consisting in a wave-like motion of the chains and platform. Every impulse applied to the bridge causes a series of oscillations of extent proportional to the impulse, which go on until they are gradually extinguished by friction; and the application of a series of impulses at intervals which are commensurable with the periodic time of oscillation of

the bridge causes the extent and the consequent straining effect of the oscillations to go on continually increasing; so that a long series of successive impulses of very small amount, occurring at regular intervals, may be sufficient to endanger or destroy a very strong suspension bridge. Such is known to be the effect of the regular tread of soldiers in marching; and, therefore, when they approach a suspension bridge, they must be instructed to break into an irregular step.

Storms of wind cause oscillations, both vertical and horizontal, which have sometimes proved very destructive.

Although the oscillation of suspension bridges cannot be wholly prevented, it may be very efficiently checked by means of a system of oblique stays, which may either be external to the framework of the bridge, or be contained within it.

As an example of a mixed system of external and internal stays may be taken that of the Niagara Suspension Bridge. In it there are 120 stays, which may be described as "guy-ropes;" they are iron ropes, each of a sectional area which is about 1-200th part of the joint sectional area of the four main cables; some of them extend obliquely downwards from the saddles on the top of the piers to the platforms; others extend obliquely downwards and sideways from the lower platform to various points of the rocks on which the piers stand. The upper, or railway platform, and the lower, or road platform, constitute respectively the top and bottom of a tubular lattice girder 24 feet broad and 18 feet deep, with timber booms and uprights diagonally braced both horizontally and vertically with iron. (See fig. 265, p. 581.)

Every well constructed suspension bridge has its platform stiffened horizontally by diagonal bracing; as to the action of which, see Article 336, p. 467. Vertical diagonal bracing is very generally used to give vertical stiffness: this will be considered more in detail further on.

In order to stiffen two suspension bridges in the Isle of Bourbon, the elder Brunel tied the platforms down to a set of inverted chains (called "counter-chains") whose total sectional area is about one-third of that of the main chains.

Suspension bridges with sloping rods are stiffer than those with vertical rods.

VI. *Bracing to resist a heavy Travelling Load.*—Various methods have been proposed, and partially tried, to enable a suspension bridge to resist the action of a heavy travelling load, such as a railway train, without undergoing more disfigurement than a girder. In order to make such methods effect their purpose completely in bridges of several bays, the chains must be made fast to piers of sufficient strength and stability, as described in p. 576.

(1.) *Auxiliary Girders.*—These are a pair of straight girders of any convenient construction (such as the plate, the zig-zag, or the lattice) hung from the chains by the suspending-rods, and supporting the cross joists of the platform. A sketch of an auxiliary girder is shown in fig. 263. It should be not merely *supported* at each end, but *fastened down*, as there are certain positions of the rolling load which tend to lift one of its ends. It should not, however, be *fixed in direction* there. In order to enable it to act with

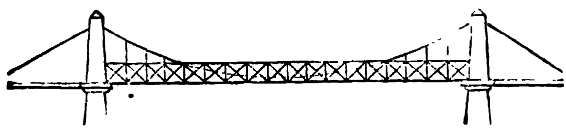


Fig. 263.

the greatest efficiency, it should be *hinged* at the middle of the span, which may be effected by making it in two halves, connected together by means of a cylindrical pin of dimensions sufficient to bear the shearing stress, which will presently be stated. The object of this is to annul the straining action which would otherwise arise from the deflection and expansion of the chain.

This precaution having been observed, the greatest bending action on the auxiliary girder will be that due to *half the rolling load*, upon a girder of *one-half of the span of the chain;* and the greatest shearing action, which will take place at the central pin, and at each point of support, will be equal to *one-eighth* of the rolling load over the whole span. That is to say, in symbols,

Let w' be the greatest rolling load per unit of span,
x, the *half-span;*
M, the moment of the greatest bending action on the auxiliary girder;
F, the greatest shearing force; then

$$M = \frac{w' x^2}{16}; \quad\quad (7.)$$

$$F = \frac{w' x}{4}. \quad\quad (8.)$$

Each half of the auxiliary girder is accordingly to be designed as if for a girder of the span x, under an uniformly distributed load of the intensity $w' \div 2$; regard being had to the fact that such load acts alternately upwards and downwards, so that each

piece of the girder must be capable of acting alternately as a strut and as a tie, under equal and opposite stresses.

If the girder is not hinged, but continuous, at the middle of the span, it should be made capable of bearing a bending action whose moment is

$$M = \frac{w' x^2}{14}; \quad \text{.........................(9.)}$$

and not to go into unnecessary nicety of calculation, the cross-section capable of resisting that moment may be continued uniformly throughout the *middle half* of the stiffening girder.*

(2.) *By Diagonally-Braced Pairs of Chains.*—This system is represented in fig. 264. In order that the two chains may be affected alike by the expansive action of heat, their curvatures should be equal; in other words, their vertical distance apart should be the same throughout the whole span. If that vertical distance be

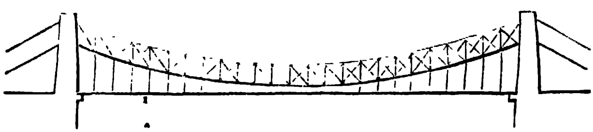


Fig. 264.

made equal to *half the depression* of each chain, no additional material will be required in the chains beyond what is necessary to support a travelling load over the whole span. The diagonal braces should be capable of acting as struts and ties alternately, under stresses computed as for an auxiliary girder. Material would be saved by this mode of stiffening, as compared with the auxiliary girder; but it would probably be less efficient and durable, as the alteration of the curvature of the chains by heat and cold would tend to strain and loosen the joints of the braces.

(3.) *By Diagonal Bracing between the Chains and Platform.*—This case is to be treated as a braced iron arch inverted, with the action of each force reversed; so that the formulæ of Article 380, pp. 567 to 570, and of Article 379, equations 3, 4, 5, pp. 563, 564, may be applied. This method of stiffening will not be efficient, unless the weight of the platform bears such a proportion to the rolling load as to prevent any suspending-rod from being subjected to thrust; and that such may be the case, the result of equation 4, p. 564, should be negative, or nothing, for each such rod. It is also open to the same objections with method (2.)

* See, on this question, the *Civil Engineer and Architects' Journal* for November and December, 1860; also, *A Manual of Applied Mechanics*, second edition, p. 375

(4.) *By Tension Ribs.*—Mr. E. A. Cowper has proposed to use, instead of flexible chains or cables, stiff wrought iron ribs, like inverted arches.

The theory of the action of the load on such "tension ribs" is precisely the same as in the case of ordinary arched ribs (see Article 374, p. 537), except that every force is reversed, tension

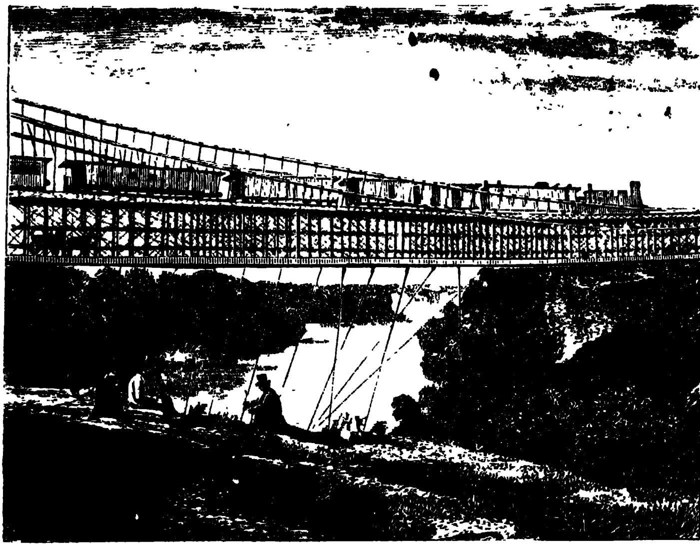


Fig. 265.—[Niagara Falls Bridge, from a Photograph.]

being substituted for thrust, and thrust for tension (if any). In order to annul the straining action of the yielding of the piers, and of changes of central deflection and temperature, those ribs should be hinged at the middle and at the points of support, in which case all the formulæ of Article 374, Case IV., p. 541, become applicable to them, with the modification stated above.

(5.) *By Straight Main Chains, with Auxiliary Suspension.*—In Mr. Ordish's form of suspension bridge, the side girders which carry the platform are supported at intervals by straight main chains, which run directly to the saddles at the tops of the piers. To preserve the approximate straightness of the main chains, they are hung at intervals from a pair of auxiliary chains of the catenarian form, which have no duty, except to support the main chains. See *The Engineer*, November and December, 1868, pp. 343 and 380.

(See p. 798.)

VII. *Table of the Principal Dimensions of some Suspension Bridges.* (See also p. 538.)

Bridge and Engineer.	Spans of Chains. Feet.	Depression of Complete Spans. Feet.	Span — Depression.	Number of Chains or Cables.	Total Effective Sectional Area. Square Inches.	Mean Net Weight of Chain per Foot of Span. Lbs	Gross Fixed Load per Foot of Span. Lbs	Thickness of Piers at Level of Platform. Feet	Thickness of Abutment at Level of Platform. Feet.	Breadth of Platform over all. Feet.
Union (round rods). (Brown.)	449	30	14·9	6	38	129	...	17·5	...	18
Menai (flat links),.. (Telford.)	570	43	13·26	16	260	880	...	29	...	28
Chelsea (flat links), (Page.)	183 348 183	29	12	4	at centre 214 at piers 230	767	...	10 (cast iron.)	77	47
Clifton (flat links, stiffened),........ (Brunel & Barlow.)	702·25	70	10·032	6	at centre 440 at piers 481	1560	3171	...	...	31
Pesth (flat links),... (Tierney Clark.)	298 666 298	47·6	14	4	at centre 480 at piers 507	1690	9892	30	140	46
Bamberg (flat links),	211	14·14	14·9	4	40·2	137	1581	15·25	...	30·5
Freiburg (wire),..... (Chaley.)	870	63	13·84	4	49	167	760	20	...	21·25
Niagara Falls (wire), (Rœbling.)	821·3	upper cables. 54 lower cables. 64	15·21 12·83	4	241·6	820	2032	...	...	24
Prague (flat links, steel), (Ordish.)..	164 492 164	about 60	about 8·2	main 12 auxiliary 2	...	...	...	16·5	114·5	32

383. **Proportion of Weight to Load in Built Bridges.**—In Article 167, p. 263, the general principles have been explained according to which the weight of a beam intended to carry a given load can be approximately determined before designing the beam. The examples there given, however, are applicable to simple beams only, in which every portion of the material directly contributes to the resistance to the bending and shearing action of the load. In large built bridges there are many parts which do not directly contribute to that resistance, but which, being necessary for the connection or staying of the parts which do so, are essential parts of the structure; and they increase its weight in a proportion which ranges in various practical examples from once and a-half to double.

To deduce from practical examples a formula for computing the probable ratio which the weight of the superstructure of a bridge of a given design will bear to the external load, the following data, from an existing bridge of similar design, are required:—

l, the span in feet;

B, the gross weight of the superstructure, either in all or per foot of span;

s_2, the factor of safety applicable to that weight (say 3);

W′, the greatest working travelling load (either over all or per foot of span) consistent with a proper factor of safety s_1 (say 6). This is not to be taken from the *actual* travelling load, but computed as follows:—Let W be the calculated breaking load; then

$$W' = \frac{W - s_2 B}{s_1} \; \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1.)$$

From these data compute the following quantity:—

$$L = l \left(1 + \frac{s_1}{s_2} \frac{W'}{B}\right); \ldots\ldots\ldots\ldots\ldots\ldots (2.)$$

then, for any other bridge of similar design and proportions, the probable proportion of the weight of the superstructure to the greatest working travelling load is given by the formula,

$$\frac{B}{W'} = \frac{s_1}{s_2} \cdot \frac{l}{L - l} \; \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.)$$

If $s_1 = 6$ and $s_2 = 3$, these formulæ become as follows:—

$$L = l \left(1 + 2 \cdot \frac{W'}{B}\right); \ldots\ldots\ldots\ldots\ldots\ldots (4.)$$

$$\frac{B}{W'} = \frac{2l}{L - l}. \qquad (5.)$$

The following are some examples of values of L:—

	L Feet.
For tubular bridges, not continuous; the depth about 1-16th of the span (as the Conway Bridge); the effective section two-thirds of the whole iron,	614
For tubular bridges, mean depth about 1-16th of the span, continuous over piers; l in the formulæ denoting the span of the greater or intermediate bays (as the Britannia Bridge),	760
Warren girder bridges, not continuous, with cast iron struts; depth about 1-15th of the span, ...	670
Warren girder bridges, not continuous, with the frame entirely of wrought iron; depth about 1-10th of the span,	900
Iron arched bridges; rise about 1-9th of the span,	630
Wire cable suspension bridge; the depression 1-14th of the span; the cables 4-10ths of the weight of the superstructure; ultimate tenacity of the wire 90,000 lbs. per square inch (as Niagara Falls Bridge),	2000

In designing railway bridges, W′ varies with the span—thus, for short spans of, say, 50 feet 2 tons, and for greater spans of, say, 100 feet $1\frac{3}{4}$ tons per lineal foot of a single line may be taken. For bridges not carrying railways, the most severe moving load may be assumed to be that of a closely packed crowd—that is, 120 lbs. per square foot of platform, so that in such cases,

$$W' = 120 \text{ lbs.} \times \text{breadth of platform in feet.}$$

For a bridge with two platforms, one carrying a road and the other a railway, those two loads are to be combined.

Section V.—*Of Various Metals and Alloys.*

384. **Lead** is used in engineering works as a covering for roofs (as to which, see Article 337, p. 468), as a material to fasten iron cramps into masonry, by filling up the cavities between them, and sometimes as a means of distributing the pressure on the beds of arch-stones (as to which, see Article 277, p. 414). As to its tenacity and heaviness, see the tables at the end of the volume. It melts at a temperature of about 630° Fahrenheit. When a fresh surface of

lead is exposed to air or water, it becomes coated in a short time with a thin grey film of oxide, which protects the metal against further oxidation, unless some acid be present capable of dissolving the oxide. (See p. 801.)

385. **Zinc** is used for covering roofs (see Article 337, p. 468), and also for coating pieces of iron to protect them against oxidation. (See Article 330, p. 462.) A fresh surface of zinc, when exposed to the air, becomes coated with a thin film of oxide, which protects the metal against further oxidation, unless an acid be present to dissolve the oxide. The coating with zinc, or "galvanizing," as it is called, of thin pieces of iron, such as sheets and wires, makes them more ductile, and a little less tenacious than before. It is effected by carefully cleansing the surface of the iron, and placing it in contact with a solution of a compound of oxide of zinc and potash; the negative pole of a galvanic battery is connected with the piece of iron, the positive pole with a plate of zinc immersed in the solution. Zinc melts at a temperature which is estimated to be about 700° Fahrenheit. At a temperature somewhat above a red heat it evaporates, and is then highly combustible.

386. **Tin—Alloys of Tin.**—Tin melts at 426° Fahrenheit. It resists oxidation better than any of the more common metals, except gold and silver. It enters readily into combination with iron; and it is by immersing well-cleansed sheets of iron in melted tin that "tin plate" or tinned iron is prepared, the iron being coated with a layer of an alloy of iron and tin, which passes gradually into pure tin at its outer surface. Although tin is very soft and ductile, most of its alloys with other metals are harder than either of the component metals.

387. **Copper.**—As to the tenacity of copper, which differs considerably according to the manner in which the metal has been treated, see the table at the end of the volume. It is diminished to about two-thirds by a temperature of 600° Fahrenheit.

Copper resists oxidation well, owing to the formation over its surface of a film of verdigris, or carbonate of copper, which protects the metal. This property, together with its great strength, makes it an useful material for fastenings of timber work and masonry in situations where iron would be rapidly oxidated, and where the cost of copper fastenings, being from six to eight times that of iron fastenings, can be afforded.

As to the use of sheet copper for covering roofs, see Article 337, p. 468.

388. **Bronze.**—Although the term "Brass" is popularly applied to all the alloys of copper, those in which it is combined with tin are more properly called *Bronze*. These compounds are harder than copper, to a degree increasing with the quantity of tin

which they contain, up to a proportion which gives the maximum of hardness.

In order that bronze may be of good quality, as regards accuracy of the figure of castings, soundness, and strength, a general principle, applicable to all alloys, should be observed in its composition,—the quantities of the ingredients should bear definite atomic proportions to each other. When this rule is not observed, the metal produced is not a homogeneous compound, but a mixture of two or more different compounds in irregular masses, shown by a mottled appearance of the castings when broken; and these masses being different in expansibility and elasticity, tend to separate from each other.

The following is a list of some of the principal alloys of copper and tin, in which the chemical equivalents of those metals are assumed to be respectively,

Copper,	63·5
Tin,	118·

Composition. By Atoms. Copper.	Tin.	By Weight. Copper	Tin.	
6	1	381	118	Bell-metal: hard and brittle: contracts in cooling from its melting point, 1-63d.
14	1	889	118	Hard bronze.
16	1	1016	118	Bronze, or gun-metal: contracts in cooling from its melting point, 1-130th.
18	1	1143	118	Softer bronze.

As the table of tenacity at the end shows, bronze, or gun-metal, is twice as tenacious as good ordinary cast iron, and as tenacious as copper in bolts, while at the same time it is harder than copper. It is much used in machinery. Lead is sometimes present in it as an adulteration, and is very injurious to its strength and durability.* (See p. 803.)

389. **Brass**, properly speaking, is the general name of the alloys of copper with zinc. They are weaker than copper or bronze, but are useful from their fusibility and ductility. The following is a table of the principal alloys of copper and zinc, in which the chemical equivalents assigned to those metals are,

Copper,	63·5
Zinc,	65·2

* For information as to the alloys of copper, tin, zinc, and lead, see a paper in the *Manchester Memoirs* for 1860, by Mr. Crace Calvert and Mr Johnson.

Composition.				
By Atoms.		By Weight.		
Copper.	Zinc.	Copper.	Zinc.	
6	1	381	65·2	Hardened copper.
4	1	254	65·2	Malleable brass.
2	1	127	65.2	Ordinary brass: contracts in cooling from its melting point, 1-60th. Tenacity, see table at end of volume.
1	1	63·5	65·2	Prince Rupert's metal: very hard.

389 A. **Aluminium Bronze** contains from 5 to 10 per cent. of Aluminium, and from 95 to 90 per cent. of copper.

Specific gravity, 7·68; heaviness, 480 lbs. per cubic foot.
Tenacity,........................ 73,000 lbs. per square inch.
Resistance to Crushing,.......132,000 lbs. per square inch.

For manganese bronze and phosphor bronze see Appendix.

(ADDENDUM to Articles 353, p. 499, and 357, p. 512.)

389 B. **Strength of Iron and Steel.**—Summary of experiments on the strength and elasticity of steel, Fairbairn (*Report of the British Association* for 1867, pp. 161 to 274.) (See also Appendix.)

	Pounds on the Square Inch
Ultimate tenacity, from..............	60,000 to 134,000
Average,............................	107,000
Modulus of rupture, from............	60,000 to 114,000
Average,............................	80,000
Crushing stress of very small blocks,	225,000
Modulus of elasticity, E, from.......	22,000,000 to 34,000,000
Average,............................	31,000,000

Malleable Cast Iron is made by the following process:—The castings to be made malleable are embedded in the powder of red hæmatite; they are then raised to a bright red heat (which occupies about 24 hours), maintained at that heat for a period varying from three to five days, according to the size of the casting, and allowed to cool (which occupies about 24 hours more). The oxygen of the hæmatite extracts part of the carbon from the cast-iron, which is thus converted into a sort of soft steel; and its tenacity (according to experiments by Messrs. A. More & Son) becomes more than 48,000 lbs. per square inch.

According to Mr. Kirkaldy, the strength of steel is greatly increased by hardening in oil.

CHAPTER VI.

OF VARIOUS UNDERGROUND AND SUBMERGED STRUCTURES.

SECTION I.—*Of Tunnels.*

390. **Tunnels in General.**—As tunnels, compared with open excavations, are an expensive and tedious class of works, and as they form inconvenient portions of a line of communication, the engineer should study to avoid the necessity for them as far as possible.

As to the setting out of tunnels, see Article 70, p. 114.

The nature of the strata through which a proposed tunnel is to pass should be carefully ascertained, not only by means of borings and shafts, but in some cases also by means of horizontal or nearly horizontal mines or *drifts*, along the intended course of the tunnel. Shafts and drifts will be further described in the ensuing articles.

The most favourable material for tunnelling is rock that is sound and durable without being very hard. Great hardness of the material increases the time and cost of tunnelling, but gives rise to no special difficulty. A worse class of materials are those which decay and soften by the action of air and moisture, as some clays do; and the worst are those which are constantly soft and saturated with water, such as quicksand and mud.

In choosing the site of a tunnel, regard should be had, not only to the nature of the material, and to the shortness and directness of the tunnel, but to the facility for getting access to its course at intermediate points by means of shafts and drifts.

The engineer should, as far as possible, avoid curved tunnels, especially those in which the curvature is so sharp or so extensive as to prevent daylight from being seen through from end to end.

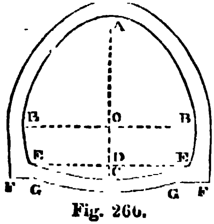


Fig. 266.

As to the figures of tunnels which require a lining of brickwork or masonry to prevent fragments of rock from falling from the roof, or to sustain the pressure of earth, and as to the strength and stability of that lining, see Article 297 A, pp. 433 to 435. Fig. 266 is an example of the elliptic form described in that article, with an inverted arch E C E at the floor. The parts F G, G F, of the base, which directly bear the side-walls and their

load, are horizontal. O is the centre of the ellipse E B A B E, B B the minor axis, A O C about three-fourths of the major axis.

Tunnels made in rock that is so sound as not to require a lining of masonry or brickwork to prevent pieces of it from falling in, may be made, if the rock is igneous, of almost any shape that is most convenient for the traffic. The elliptical or horse-shoe form already described, is, however, generally adopted for the sides and top, the floor being level. In stratified rocks, the strongest form for the roof is that of a pointed arch; though a flat roof has been used where the rock consists of thick layers, and has few natural joints.

In ordinary tunnels, measured within the masonry or brickwork, the dimensions of most common occurrence are—

	Height.	Width.
For single lines of railway,	20 ft.	15 ft.
For double lines of railway,	24 ft.	from 24 ft. to 30 ft.
For navigable canals,......	from 14 ft. to 30 ft.	from 14 ft. to 30 ft.

The *smallness* of tunnels for water-conduits and drains is limited by the least dimensions of the space in which miners can work efficiently; that is, about $4\frac{1}{2}$ feet high and 3 feet wide.

Authorities, Simms, *On Practical Tunnelling*; Drinker, *On Tunnelling* (New York), Rziha's *Tunnelbaukunst*.

391. **Shafts or Pits.**—Shafts or pits are sunk for three purposes: to ascertain the nature of strata to be excavated, as already mentioned in Article 187, p. 331, when they are called *trial shafts*; to give access to a tunnel when in progress, for the purpose of carrying on the work, removing the material excavated, admitting fresh and discharging foul air, and pumping out water, when they are called *working shafts*; to admit light and fresh air at intervals to, and remove foul air from, a tunnel when completed, when they are called *permanent shafts*.

I. *Trial Shafts* are in general sunk at or near the centre line of the proposed tunnel. Their transverse dimensions are fixed mainly with a view to convenience in sinking them. Six feet is an ordinary diameter for a round trial shaft, six feet by four are ordinary dimensions for rectangular shafts. The shape is regulated by the material to be used in lining the shaft, being rectangular in timbered shafts, and cylindrical in those that are *steined* or lined with stone or brick.

The number and distance apart of trial shafts are to be determined after previous boring, in the same manner as for a deep cutting (Article 187, pp. 331, 332); that is to say, no general rule can be laid down on the subject; but the engineer must, to

the best of his judgment, sink such shafts as are necessary in order to give him an accurate knowledge of the strata to be excavated.

II. *Working Shafts* may be either rectangular or round. Their usual transverse dimensions range from 6 feet to 9 feet; the greater diameter is advantageous, because of its admitting of large quantities of material being raised and lowered at a time. Their distance apart varies, in ordinary cases, from 50 to 300 yards. In some cases, however, it has been found necessary to place them as close as 20 or 30 yards apart, for the purpose of discharging foul air; while in other cases the height of the ridge to be tunnelled through has rendered the sinking of shafts impracticable for very long distances. An extreme example of the last case is the tunnel through Mont Cenis (see p. 596), which is 7·59 miles long, and which has been excavated entirely from the two ends, without the aid of shafts.

The range of working shafts of a tunnel may lie either along its centre line, or in a line parallel to the centre line, at an uniform distance to one side. When the latter system is adopted, the object is to keep the shafts clear of the excavation and building of the tunnel, with which they are connected by cross drifts.

When a working shaft is to be used in order to drain the tunnel of water as the work proceeds, it is sunk to such a depth below the bottom of the excavation as to form a sufficient reservoir for water, called a "*sump*," from which the water is raised by a windlass and buckets, or by a pump. The most convenient form of bucket is one that is hung in a stirrup by a pair of trunnions whose axis nearly traverses the centre of gravity of the bucket. When lowered, the bucket is held upright by a catch; and after it has been raised, the removal of the catch allows it to be easily tilted over, in order to discharge the water.

III. *Permanent Shafts* are in general working shafts that have been made permanent parts of the structure, the brick lining of each being supported on a permanent *curb*, or suitably formed ring of brickwork, or of cast iron, surrounding a circular orifice in the roof of the tunnel. The top of each shaft is protected by being surrounded with a wall, and covered with a grating.

Permanent shafts are occasionally met with of a diameter as great as, or greater than, that of the tunnel. For example, the shafts at the ends of the Thames tunnel are 50 feet in diameter; the tunnel itself consisting of a pair of archways, each 14 feet in clear width, and the entire width of passages and brickwork being $37\frac{1}{2}$ feet.

IV. *Sinking Shafts in sound rock* is performed simply by the operations of blasting and quarrying, as already described in Article 207, p. 344. In order to be safe from the effects of

explosions, the workmen should ascend to a height of 50 or 60 feet above the bottom of the shaft (if it is so deep), before each blast is fired.) The noxious fumes produced by the powder may be partially dispersed or absorbed by dashing in a bucket of water; but a more efficient plan of ventilation, especially in deep shafts, is either to extract the foul air through a sheet iron tube leading up to a furnace or to an exhausting fan, or to blow fresh air down by means of a fan through such a tube.

Ventilating apparatus is indispensable when foul air (such as carbonic acid gas, or "choke damp") or inflammable gas ("fire damp") is disengaged from the strata that are traversed by the shaft.

When water flows into the shaft, it is to be collected at the bottom in a "sump" or well of smaller diameter than the shaft, and raised by buckets, or by pumping, either to the surface of the ground or to some drift through which it can be discharged.

V. (*Sinking Timbered Shafts.*—A shaft sunk through soft materials, or through loose rock, must be lined with timber, masonry, or brickwork.)

The principal pieces in the timbering of a shaft, as well as in the timbering of drifts, tunnels, and underground excavations in general, may be distinguished into *props*, which are struts or posts, either vertical or raking, and usually of round timber; *sills* and *bars*, being horizontal pieces, sometimes round and sometimes squared; and *cleading* or *boards*. Props are combined with sills or bars into framework simply by abutting joints at their ends, which are made fast in their places by the aid of spikes called "*brobs*," of the shape shown in fig. 267, and usually about 6 inches long. Fig. 268 represents the foot of a prop resting on a sill, and made fast with four brobs, of which three are shown. The shape of the head of a brob enables it to be knocked out as easily as it is driven in.

Fig. 267.

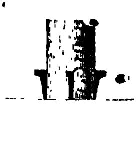


Fig. 268.

Fig. 269 is a section of a square-timbered shaft of about 9 feet square. The timbering consists of horizontal square frames or "*settings*," one at every six feet of depth or thereabouts, each made of four square sills of 12 inches × 12 inches, supported by round props of 8 or 9 inches diameter, and clad outside with vertical "*poling boards*" of 3 inch deal. The shaft having been sunk and timbered as far as the earth will stand for a time vertical, the further sinking is effected as follows:—In the centre of the bottom of the shaft a small pit is dug, at the bottom of which, at A, is laid a small platform of boards; then by cutting notches in the sides of the pit, "*raking props*," such as those shown by dotted lines, are inserted;

their lower ends abutting against a "*foot-block*" at A, and their upper ends against the lowest setting, so as to give it a temporary support. The pit is then enlarged to the dimensions of the shaft above; vertical poling boards are set up against its sides, with their upper ends behind the temporarily supported square setting, and their lower ends behind a new square setting, laid on the bottom of the excavation; vertical props are inserted between those settings, and made fast; the raking props and their foot-blocks are taken away; a new small pit is dug, and so on as before. Care should be taken that the earth presses firmly against the poling boards. Should streams of water come in through the chinks between the boards, the tendency of those streams to carry with them particles of sand, and so to leave cavities in the earth, may be counteracted by stuffing straw behind the boards.

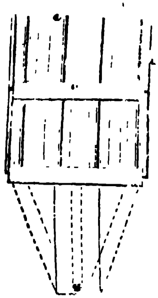


Fig.

VI. *Sinking Stone or Brick-lined Shafts* (which are usually cylindrical) may be effected in two ways; by "*underpinning*," or by a "*drum-curb*."

To sink a shaft by *underpinning*, it is first dug as deep as the earth will stand vertical. At the bottom of the excavation is laid a "*curb*;" that is, a flat ring, whose internal diameter is equal to the intended clear diameter of the shaft, and its breadth equal to the thickness of the brickwork (usually 9 inches). It is made of oak or elm planks 3 or 4 inches thick, either in one layer fished at the joints with iron, or in two layers breaking joint, and spiked or screwed together. On this, to line the first division of the shaft, a cylinder of brickwork is built in hydraulic mortar or cement. In the centre of the floor is dug a small pit, as described in Division V. of this article, at the bottom of which a platform and foot-blocks support raking props, which are inserted to give temporary support to the curb with its load of brickwork; the pit is enlarged to the diameter of the shaft above; on the bottom of the excavation is laid a new curb, on which is built a new division of the brickwork, giving permanent support to the upper curb; the raking props and their foot-blocks are removed; a new pit is dug, and so on as before. Care should be taken that the earth is firmly packed behind the brickwork, and that the shaft is carried down truly vertical.

A *Drum Curb* (fig. 270), which may be made of timber or of cast iron, consists essentially of a flat ring for supporting the brickwork, and of a vertical hollow cylinder or drum, of the same outside diameter as the brickwork, supporting the ring on its upper edge, and bevelled to a sharp edge below. The drum may be strengthened

if necessary by an additional ring, and its connection with the rings made more secure by brackets, as shown in the figure.

When the shaft has been sunk as far as the earth will stand vertical, the drum-curb is lowered into it, and the building of brick cylinder commenced, care being taken to complete each course of bricks before laying another, in order that the curb may be equally loaded all round. The earth is dug away from the interior of the drum; and this, together with the gradually increasing load of brickwork, causes the sharp lower edge of the drum to sink into the earth; and thus the digging of the shaft at the bottom, the sinking of the drum curb, and the brick lining which it carries, and the building of the brickwork at the top, go on together.

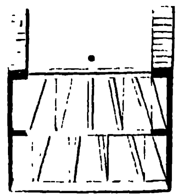


Fig. 270.

Great care must be taken so to regulate the digging that the shaft shall sink vertically.

Should the friction of the earth against the outside of the shaft at length become so great as to stop its descent, before the requisite depth is attained, a smaller shaft may be sunk in the interior of the first shaft. A shaft so stopped is said to be "earth-fast."

VII. *Temporary Support of Working Shafts.*—When a working shaft is sunk in the centre line of an intended tunnel, it is obvious that the completion of the excavation for the tunnel will remove the support from below the lining of the shaft, which support will only be replaced when the arching of the tunnel is completed.

There are two modes of giving temporary support to the shaft, from below and from above.

Support from below is given, if the ground is solid enough, by means of a pair of strong parallel sills, say 15 inches square, and 10 feet longer than the intended span of the tunnel. Each of these is sent down the shaft in three pieces, which are inserted into small horizontal drifts running at right angles to the line of tunnel, about 3 or 4 feet above its intended roof, and are there scarfed together. The drifts are then rammed up. The distance between the two sills is equal to the clear width of the shaft. They support a square frame, which supports the lowest curb of the part of the shaft to be carried.

Should the material be too soft to admit of this mode of support, the two sills (each of which may now be in one piece) are to be laid on the surface of the ground over the mouth of the shaft across the line of tunnel, and somewhat closer together than the width of the shaft. The lower end of the shaft is carried by a strong wooden frame, which is hung from the two sills by means of four wrought iron suspending-rods or chains.

The part of the shaft thus temporarily supported is generally lined with brick: the part below the temporary support is lined with timber, which, is removed in the course of the excavation of the tunnel.

392. **Drifts, Mines, or Headings,** are small horizontal or inclined underground passages, made in order to explore the strata in the line of an intended tunnel, to drain off water, and to facilitate the ranging of the line and levels and setting out of the works (see Article 70, p. 114), the access of the workmen, and the transport of materials; and for the last-mentioned purpose they are often furnished with small temporary railways.

I. *Positions of Principal Headings.*—The working shafts of a tunnel are almost always connected together by means of a heading, which accordingly runs either along or parallel to the centre line of the tunnel. In some cases the heading runs along the centre line, while the working shafts lie at one side, and are connected with the main heading by cross headings.

When a tunnel runs through a steep hill, near or parallel to one of the sides of the hill, cross headings opening above ground at the hillside may be used instead of working shafts; but such cases seldom occur.

In tunnelling through soft and wet ground the most convenient level for the principal heading is at or near the bottom of the tunnel. In hard and dry materials it may be placed near the roof. Other positions will be mentioned farther on.

II. *The least Dimensions of a Heading* in which miners can conveniently work are about 3 feet broad and $4\frac{1}{2}$ or 5 feet high.

III. *Headings in Solid Rock* are driven by blasting and quarrying, as to which, see Article 207, p. 344.

Machinery is employed in large works for driving headings, and an early application was made at the tunnel through Mont Cenis. It consisted of a number of horizontal jumpers, driven at the rate of about 250 blows per minute, by means of air compressed to five atmospheres, and conveyed into the mine through pipes. The air was supplied and compressed by hydraulic machinery near the outer end of the mine. Since that time great improvements have been made in the machinery employed for excavating in rock. The jumper has been replaced by the drill: these drills have sharp cutting edges, and are driven at a high rate of speed by air pressure. Various kinds of powerful explosives (see p. 348) can now be obtained. The difficulty of ventilation in long tunnels can be largely got over through the use of electricity as a motive power for the traffic. The summit of the Mont Cenis Tunnel is 4,245 feet above sea level, that of the St. Gothard 3,786 feet, and the Simplon 2,313 feet. (See also pp. 590, 596, and 810.)

IV. *Timbered Headings.*—Headings in loose and soft materials are lined with timber, the principal parts of the timbering being, as in other cases of the timbering of excavations, horizontal pieces, props, and poling boards. Fig. 271 is a longitudinal section of a heading in earth proceeding in the direction shown by the arrow. The frames, or "settings," are placed at from 2 to 3 feet apart, and are made of round timber 5 or 6 inches in diameter, so that the pieces can be easily handled by one man. The section shows the ground-sills resting in grooves cut in the floor, the props standing on them, the upper horizontal pieces called "cap-sills," resting on the props, and the poling boards driven between the settings and the sides and top of the excavation. These boards are usually from $\frac{3}{4}$ inch to an inch thick. In running sand and other soft and wet materials, poling boards are laid under the bottom sills also, so as completely to enclose the heading; and straw is packed behind the boards, to keep sand from running in through the chinks. The operations of carrying the heading forward are as follows:—Drive a set of poling boards forward into the earth, between the last setting and the forward ends of the last set of poling boards; then excavate the earth within the new set of boards, and insert a new setting, and so on.

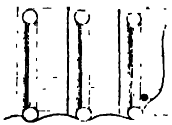


Fig. 271.

V. *Precautionary Borings.*—In driving a mine through ground in which it is possible that cavities containing large quantities of water may be encountered, borings ought to be carried forward both directly and obliquely in advance of the mine, in order that the neighbourhood of such cavities may be ascertained in time to guard against sudden outbreaks of water into the mine, and that the water accumulated in them may escape by degrees and without danger through a bore-hole. This precaution is especially necessary in approaching old pits, mines, or tunnels, which are very generally found to be full of water.

VI. The *Cost and Labour of Mining*, all things included, such as blasting, timbering, removing water, lights, temporary rails and waggons, &c., vary from five times to twenty times the cost and labour of excavating the same quantity of the same material in the open air.

393. **Tunnels in Dry and Solid Rock** are in general excavated by driving a heading immediately below the intended roof of the tunnel, from which heading the excavation is extended sideways and downwards by blasting and quarrying.

These operations require labour to the extent of from three-fourths

of a day's work to three days' work of a miner per cubic yard of rock, according to its hardness, being considerably more than is required in the open air.

The Mont Cenis and St. Gothard Tunnels are about 7 and 9 miles long respectively, and were cut through the rock by means of drills driven by air compressed by water power. On account of the ends of the Mont Cenis Tunnel being visible from the summit, the ranging of the centre line was accomplished by means of a telescope set in a tower. The St. Gothard Tunnel was set out by connecting the ends by a series of triangles, the intervening ground preventing a direct observation, as in the former case.

The Mersey and Severn Tunnels, completed some years ago, are about $2\frac{1}{4}$ and $4\frac{1}{2}$ miles long respectively, of which about one-half in each case is under water. The Mersey Tunnel is 26 feet wide and 23 feet high; it is cut through sandstone rock, and lined with brick in cement. The Severn Tunnel passed through sandstone and shale; it is 26 feet wide, the lining being brick in cement 2 to 3 feet in thickness. (See p. 810.)

394. **Tunnels in Dry Fissured Rock** require brick or stone arching within, to guard against the fall of portions of the roof. The most convenient way to make them is in general to commence at a heading running along close below the roof of the excavation; to extend the excavation sideways and downwards to the floor at each side of the tunnel, leaving a wall of rock standing in the middle. This wall is used as a pier to support temporary props (should such be required) for the roof of the excavation, and also to support the centres for the arching, which is carried forward as close behind the excavation as the convenience of working will admit. When the arching is complete, and the centres struck, the central wall of rock is cut away.

All hollows between the brickwork and the rock should be carefully filled with concrete.

The labour of executing brickwork in tunnels (including cost of lights) is about double of that of executing the same quality of brickwork above ground.

395. **Tunnels in Soft Materials,** whether such as are soft from the first, or such as become soft by exposure to air and moisture, like some kinds of clay, require timbering to support the sides and top of the excavation, constructed on the same principles with that of headings.

In such tunnels a principal heading is in general required at the level of the floor, for purposes of drainage.

The excavation of the tunnel is carried on in various ways; that which will here be described is the method of which a detailed account is given by Mr. Simms in his *Practical Tunnelling*, as having been practised at Blechingley tunnel and Saltwood tunnel; the former in blue shale, and the latter in sand.

The tunnel is executed in *lengths*, each of about 12 or 15 feet. These are designated as follows, in the order in which they are executed:—

Side lengths, on each side of a working shaft.

Leading lengths, in prolongation of the tunnel from the side lengths.

Junction lengths, where two portions of the tunnel meet midway between two shafts.

Shaft lengths, directly under the working shafts.

The first operation in commencing a side length, leading length, or junction length, is to drive a heading at the roof of the excavation, whose roof must be $1\frac{1}{2}$ or 2 feet above the rounded top of the brickwork.

From that heading the excavation is extended sideways and downwards by a process exactly like that of driving a heading, as shown in fig. 272, which is a cross-section of the excavation, after it has been extended a short distance to the right of the top heading. The earth is supported by poling boards, which are supported by strong horizontal timbers called *bars*, 8 or 10 inches in diameter. The after ends of these bars are supported,—

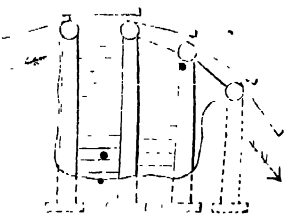


Fig. 272.

In side lengths, by props resting on the framework of the working shaft;

In all other lengths, by the top of the arch of the previous length;

And they are kept asunder by four or five short struts between each pair. The forward ends of the bars rest on props, each of which stands on a foot-block.

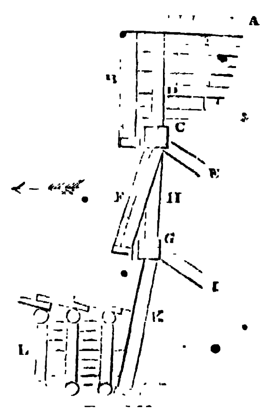


Fig. 273 is a longitudinal section, showing the timbering of the excavation of a length of the tunnel when complete; the pieces being lettered in the order in which they are put in.

A are bars already mentioned, covered with poling boards.

B, props resting on foot-blocks, and covered with poling boards. When the excavation has been carried down to the level of these foot-blocks, there is inserted—

C, a strong sill (say 13 inches square), sent down in two pieces and scarfed together. It extends completely across the excavation, and $1\frac{1}{2}$ or 2 feet into the earth at each side; and at first rests on the earth.

D, props inserted so as to rest on the sill C and support the bars A. Places are now cut to receive

E, struts, 2 or 3 in number, 10 inches in diameter, or thereabouts, whose forward ends abut against the sill C, and their backward ends in side lengths against the timbering of the shaft, and in other lengths against notches in the completed brickwork.

The excavation being by degrees carried down, there are inserted—

F, raking props below the sill C, standing on foot-blocks, and covered in front with poling boards. When the excavation has been carried down to the level of the foot-blocks, there is inserted—

G, a lower sill, similar to C; and this is ultimately supported and kept in its place by struts I and raking props K, in the same manner with C.

L is part of the bottom heading.

The bottom of the excavation is formed with great accuracy to receive the invert, or inverted arch, which forms the base of the brickwork, the levels being set out as described in Article 70, p. 116. The invert and side walls are built according to moulds, as described in Article 252, p. 392; and the arch of the roof upon centres, consisting of three ribs under each length. The best centres have ribs of iron, with screws under each laggin. The centres are usually supported on cross sills, which are themselves supported partly by posts resting on the floor, and partly by their ends being inserted into holes in the side walls, which are built up after the centres are struck.

After the brickwork of a length has been built, most of the crown bars which lie above the arch can be pulled forward so as to serve for the next length; those which resist this must be left. All spaces between the brickwork and the earth must be carefully rammed up.

The following was the distribution of the cost of Blechingley tunnel, according to Mr. Simms:—

MATERIALS.		Per Cent
Bricks,	$30\frac{1}{2}$	
Cement,	11	
Timber,	$11\frac{1}{6}$	
Ironwork,	$2\frac{1}{2}$	
Miscellaneous,	$6\frac{1}{2}$	
Carried forward,		62

Brought forward,..		62
Labour.		
Mining—Shafts, heading, &c.,............	$13\frac{1}{4}$	
,, Tunnelling,..................	$5\frac{3}{4}$	
		19
Brickwork,..		12
Miscellaneous Expenses,		
Such as tunnel entrances, culvert, machinery, buildings, inspection, &c., ...		7
		100

The total cost per yard forward was about £72; the clear dimensions of the tunnel being 21 feet × 24 feet, and the brickwork from 1 foot $10\frac{1}{2}$ inches to 3 feet thick.

The form of cross-section is that already given in fig. 266, p. 588.

396. **Tunnel Fronts—Drainage.**—A tunnel front consists of spandril walls above the arch, and wing walls at each side, like those of a bridge. To secure the end of the arch against the tendency of the slope of earth above it to push it outwards, it may be tied back by longitudinal iron rods to a horseshoe-shaped curb of cast iron, built into the brickwork at a distance back from the front about equal to the height of the tunnel.

A tunnel which has no invert may be drained by means of a pair of side drains, like a cutting; but where there is an invert, the main drain should be a central culvert, of which the invert itself may form the floor.

A catch-water drain should divert the surface water which might otherwise flow over the tunnel front.

397. **Tunnelling in Mud.**—The celebrated tunnel of the elder Brunel under the Thames consists of a rectangular mass of brickwork laid in cement, 37·5 feet broad and 22 feet high, containing a pair of parallel horseshoe archways, each 14 feet span and 17 feet high, which are connected together by small cross archways at intervals. The least thicknesses of the brickwork are, at the crown of the arch $2\frac{1}{2}$ feet, at the base of the invert $2\frac{1}{2}$ feet, at the sides 3 feet, in the central wall $3\frac{1}{2}$ feet. The whole mass of brickwork rests on a base of elm planks 3 inches thick.

In driving this tunnel, the place of the timbering of the excavation described in Article 395, was supplied by a machine called a "shield," which was pushed on in advance of the brickwork at a distance of about eight feet. The shield was of the same dimensions with the mass of brickwork. It consisted of twelve

equal and similar divisions standing vertically side by side, and capable of being pushed forward to a short distance independently of each other. Each division consisted of a cast- and wrought-iron frame about 3 feet broad (to allow a small space between the frames), containing three stages for workmen. It had two cast-iron feet, resting on the floor of elm planks; on these feet it was supported by a pair of hinged legs of lengths adjustable by screws. It had an iron roof extending back to the brickwork, and a pair of jack-screws at the top and bottom, abutting against the front end of the brickwork, to push it forward. The several frames were connected together by hinged arms, nearly vertical, to enable them to afford support to each other when required. The spaces at each side of the shield, extending back from the face of the excavation to the brickwork, were guarded by iron plates. Each frame had in front of it, extending from top to bottom, a range of poling boards; each poling board was 3 inches thick and 6 inches broad, and was pressed against the material in front by a pair of small jack screws abutting against the frame. A railway now passes through this tunnel

Iron tunnelling under air pressure is adopted in soft ground. The rings are of cast-iron segments, bolted together at flanged joints, the latter being made water-tight. To insure tightness, the joints are sometimes iron to iron, and in other cases, wooden wedges, rust, or cement have been used. The work is done under air pressure, and "air locks" have to be provided for the workmen. The amount of pressure will vary with the conditions outside the tunnel. Thus, if the work is below the bed of a tidal river, the pressure required to keep back the water will vary with the height of the tide.

The air locks are small chambers made of steel plate, with suitable doors and fittings for the supply of air and water and passage of materials.

The diameters of some of the more important iron tunnels are as follows:—

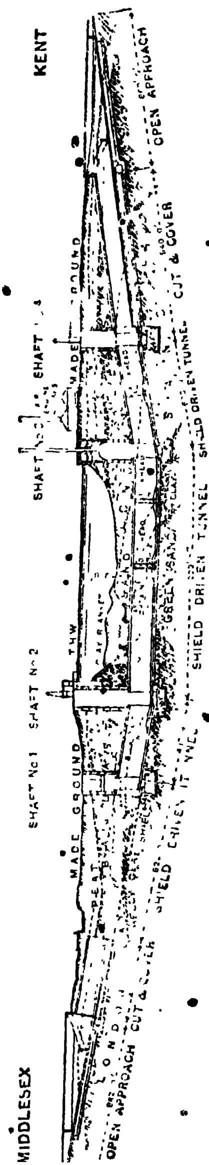
Blackwall Tunnel, London, . . .	5 feet internal diameter.
Waterloo and City Tunnel, London, .	12 " "
St. Clair Tunnel, U.S.A., . . .	19 feet 10 inches internal dia.
Glasgow Harbour Tunnel, . . .	16 feet internal diameter.
Glasgow District Subway, . . .	11 " "

The air pressure under which the work can be carried on with safety depends on various conditions—about 30 lbs. per square inch being a possible limit, although higher pressures have obtained. At pressures of 25 to 30 lbs. the working hours should be less than with lower pressures, and the length of time allowed the men to pass from the working pressure to the atmospheric pressure should be increased. (See also pp. 601 and 801.)

Modern Typical Examples.—Since the Thames tunnel was made below the river in the early part of the present century, the difficulties and the doubtful financial results of such a means of cross-river communication have deterred engineers and investors from adopting this method of communication until comparatively recent years, when the growing demand for means of transit and the improvements in the construction of materials and machinery, with the reduced cost at which these could be supplied, have caused the evident advantages of such a method of cross river communication to be once more taken up, and now we are enabled to enumerate several undertakings of this kind which have been brought to a successful issue by the skill, resource, and perseverance of the engineers, contractors, and workmen engaged.

Of the tunnels which have in this way been constructed special mention may be made of the Tower Subway, the Blackwall Tunnel, the Hudson River Tunnel (not yet completed), and the St. Clair Tunnel. (For dimensions of these and other tunnels, see p. 600.)

The Tower Subway, which crosses underneath the River Thames, near the Tower, was constructed in 1868; its diameter is 6·7 feet, and it is the first tunnel which was lined with cast-iron rings. A shield was used; but as the tunnel was wholly driven through the London clay, compressed air was not required. In many cases the material to be tunnelled is so soft that it would flow into, and impede the working if not kept back by means of air forced into the workings. This method has been frequently adopted

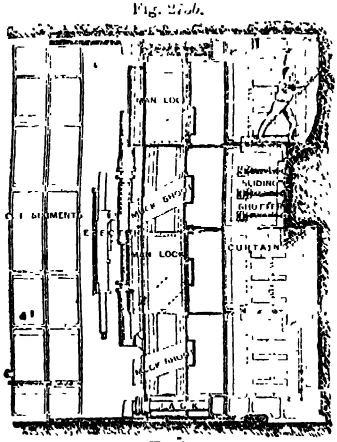


Blackwall Tunnel Longitudinal Section.

in the sinking of cylinders for the piers of bridges (see pp. 607 and 801); under such circumstances the difficulties are very much increased. In the works at the Hudson River and St. Clair Tunnels, compressed air was used, and lately at the Blackwall Tunnel the same system had to be adopted. This tunnel is the largest yet made, lined with cast iron, and worked by a shield and compressed air.

Some other works in this country have also been carried out in this manner, notably the City and South London Electric Railway, the Glasgow District Subway, the Mound Tunnels, Edinburgh, and the Glasgow Harbour Subway. The diagram on p. 600*a* is from a paper read by Mr. E. W. Moir, M.Inst.C.E., on the Blackwall Tunnel, before the Society of Arts, 1896, and graphically shows in longitudinal section this important undertaking. As described in the paper, the main features of construction may be enumerated as follows:—Four shafts having been sunk in the positions shown on the figure, the tunnel was then driven, partly through sand and clay beneath the river and for a considerable length at an incline under the banks.

Fig. 273*b*.



BLACKWALL TUNNEL SHIELD (PEARSON'S PATENT) LONGITUDINAL SECTION

A shield of a special construction was used (see fig. 273*b*) and of the following dimensions:—Diameter, 27 ft. 9¼ ins.; length, 19 ft. 6 ins. Four thicknesses of ⅝ in. steel plate were used for the outer skin, the face being divided by girders into working chambers. There are three horizontal girders fitted carrying the cutting edges. Air locks were provided for the passage of the workers, and similar arrangements acted as shoots for the excavated material. By means of hydraulic jacks the shield was pressed forward, and the cast-iron segments were fitted in behind, and the space left vacant through the different diameters of the shield and tunnel was grouted with lime. In this way the construction of the tunnel proceeded at the rate of about 252 feet per month when under the Thames, except at a part where a bed of gravel was met with, and, the compressed air escaping through the bed of the river, caused delay. At one point the crown of the tunnel came within a few

feet of the river bed, and quantities of clay, as in other cases of a like nature, were thrown in to keep the working tight. One of the most important arrangements in the construction of such tunnels is the air lock through which the men pass to work. It has been proved that the passage from the high to the low pressure on coming out should be gradual. (See also p. 801.) It was found, at the works of the Hudson Tunnel, that where the air pressure was as much as 35 lbs. per square inch numerous cases of paralysis occurred; but that by placing the sufferers in a specially prepared compressed air chamber, and *gradually* reducing the pressure, and so equalising it during an interval of about half an hour, the men were quite restored.

SECTION 11.—*Of Timber, Iron, and Submerged Foundations.*

398. **General Principles—Submerged Foundations.**—The general principles already explained with reference to ordinary foundations, viz., that the base should be as nearly as possible perpendicular to the resultant pressure, and that the centre of pressure should not deviate from the centre of figure of the base beyond certain limits, are applicable to the foundations considered in the present section also. The mathematical expression of those principles has been given in Articles 236, 237, pp. 377 to 380.

In calculations respecting the stability of structures whose foundations are submerged in water, it is to be borne in mind that the pressure of the water on the immersed part of the structure has the same effect as if the weight of that part were diminished by an amount equal to the weight of an equal volume of water; that is, as if the heaviness of the immersed part of the structure were diminished by 62·4 lbs per cubic foot. (See Article 107, Division IV., p. 165.)

399. **Foundations on Timber Platforms** are employed where the ground is too soft and wet for the expedients mentioned in Article 239, p. 381. The best European timber for such platforms is elm or oak. Beams of from 10 inches to 1 foot square are laid about 3 feet apart, in two layers, crossing each other so as to form a grating, the space between them is filled with concrete, and above them is laid a layer of planking, 3 or 4 inches thick, on which the building rests. Another mode of constructing such platforms is to lay several layers of planks and pin them together. In order that timber platforms may be durable, they should be constantly wet.

400. **Foundations on Iron Platforms.**—As a practical example of a platform of this kind, may be cited the cast iron invert which was substituted for a stone invert in a lock at Grangemouth. The lock is 30 feet broad; the depth of water 18 feet 6 inches; the

side walls about 8 feet thick and $20\frac{1}{2}$ feet high; the invert in question consists of a series of trough-shaped cast iron girders, lying close together side by side, and bolted to each other through their vertical sides; each of them is 2 feet broad, 21 inches deep at the springing, 12 inches deep at the centre of the invert, and 2 inches thick, each of their vertical sides has a flat horizontal flange at the top, 4 inches broad. The trough-shaped interior of each girder is filled with concrete, covered with a layer of bricks laid in cement.

401. **Short Piles** are driven in order to compress and consolidate the soil. They are usually of round timber, from 6 to 9 inches in diameter, and from 6 to 12 feet long, and are planted as close to each other as is practicable without causing the driving of one pile to make the others rise. The outside row of piles should be driven first, then the next within, and so on to the centre. The mass of consolidated soil and piles thus produced may be regarded, as respects the relation between its bulk and the load that it can bear, in the same light as if a trench had been dug of the same volume, and filled with a stable material; as to which, see Article 239, p. 381. On the top of the piles may be placed either a platform, a layer of concrete, or both.

402. **Bearing Piles** act as pillars, each supporting its share of the weight of the building. They may either be driven through the soft stratum until they reach a firm stratum and penetrate a short distance into it; or, if that be impracticable, they may be supported wholly by the friction of the soft stratum. It appears from practical examples that the limits of the safe load on piles are as follows:—

For piles driven till they reach the firm ground, 1,000 lbs. per square inch of area of head.

For piles standing in soft ground by friction, 200 lbs. per square inch of area of head.

The diameters of long piles range from 9 inches to 18 inches, and should never be less than 1-20th of the length. Their distance from centre to centre averages about 3 feet, and is seldom less than $2\frac{1}{2}$ feet.

The best material for them is elm, which should be chosen as straight-grained as possible. The bark should be removed, and knots or rough projections smoothed off. (See p. 416.)

Piles should be driven with the butt or natural lower end of the timber downwards. It is roughly sharpened to a point whose length is from $1\frac{1}{2}$ times to twice its diameter; and should stones or other hard materials occur in the strata to be pierced, the point must be fitted with a "shoe" of cast or wrought iron, fastened on with spikes. The weight of these shoes averages about 1-100th part of that of the piles.

To prevent the head of a pile from being split or bruised by the

blows of the "*ram*" used in driving it, it is bound with a wrought iron hoop.

Pile-driving engines are of various kinds. The simplest is the "*ringing engine*," in which the ram, weighing about 800 lbs., and moving between timber guides, is attached to one end of a rope which passes over a pulley. The other end of the rope branches out into a number of smaller ropes, each held by a man, in the proportion of one man for each 40 lbs. weight of the ram, or thereabouts. The men, pulling all together, lift the ram 3 or 4 feet, and on a given signal, let go all at once, so as to let it fall on the head of the pile. It is found that they work most effectively when, after every 3 or 4 minutes of exertion, they have an interval of rest; and under these circumstances they can give about 4,000 or 5,000 blows per day.

In the "*monkey engine*," the ram, weighing about 400 lbs., and held by a staple in a pair of tongs, is drawn up 10 feet, 15 feet, or higher if necessary, by means of a windlass; at the top of the lift the handles of the tongs come in contact with two inclined planes which cause them to let the ram fall; the tongs are then lowered, and have jaws so shaped that on reaching the staple at the top of the ram they lay hold of it again. The windlass may be driven by men, horses, or steam power.

The steam hammer is sometimes used for driving piles; and also an engine somewhat on the same principle, in which the ram is lifted by the pressure of compressed air. In such machines rams of great weight are sometimes used, such as 1 ton, or a ton and a-half.

Piles may be driven in a direction either vertical or raking, according to the position of the guides between which the ram slides. That direction should be parallel to that of the pressure which they are to resist.

When the head of a pile is to be driven below the reach of the stroke of the ram, the blow is transmitted from the ram to the pile by means of an intermediate short post of timber called a "*punch*," or "*dolly*."

According to some of the best authorities, the test of a pile's having been sufficiently driven is, that it shall not be driven more than *one-fifth* of an inch by *thirty blows* of a ram weighing 800 lbs. and falling 5 feet at each blow; that is to say, by a series of blows whose total mechanical energy amounts to

$$30 \times 800 \times 5 = 120{,}000 \text{ foot-pounds.*}$$

* The following formulæ show the relation between the blow required to drive a pile a given depth, and the greatest load that it will bear without sinking further, supposing it to be supported by an uniformly distributed friction against its sides.

Piles are *drawn*, when required, by means of the hydraulic press.

When a firm stratum, into which the points of a set of piles are driven, underlies a stratum so soft that their lateral stability is doubtful, a mass of loose stones may be thrown in round them to give them the steadiness which they want.

After the driving of a set of piles has been completed, their heads are to be sawn off to the height required for the support of the platform.

The soft ground round the tops of the piles is then to be scooped out to a depth which in ordinary cases ranges from 3 to 6 feet, and the space filled with hydraulic concrete, laid in layers not exceeding 1 foot deep.

The platform supported by the piles consists of a grating of beams of 10 or 12 inches square, called *string-pieces* and *cross-pieces*, half-notched into each other over the heads of the piles, to which they are fixed by treenails, and covered with planking 3 or 4 inches thick. The spaces between the beams of the grating are to be filled with hydraulic concrete. The beams on the top of the outermost rows of piles are usually made so deep that their upper surfaces are flush with that of the planking, which is *rabbeted* into them; that is, sunk in a groove. Those beams are in this case called the *capping*.

Piles may be driven into rock by first jumping holes in it of a little less diameter than the piles.

For *cast iron* piles, the best form is that of a tube. To prevent their being broken by the blows of the ram in driving them, a

Let W be the weight of the ram.
h, the height from which it falls.
x, the depth through which the pile is driven by the *last* blow.
P, the greatest load it will bear without sinking farther.
S, the sectional area of the pile.
l, its length.
E, its modulus of elasticity.

Then the energy of the blow is thus employed:—

$$W\,h = \frac{P^2\,l}{4\,E\,S}\ \text{(employed in compressing the pile)} + P\,x\ \text{(employed in driving it)};$$

and consequently,

$$P = \sqrt{\left(\frac{4\,E\,S\,W\,h}{l} + \frac{4\,E^2\,S^2\,x^2}{l^2}\right)} - \frac{2\,E\,S\,x}{l}.$$

Piles are usually driven until P, as computed by this formula, is between 2,000 and 3,000 lbs. per square inch of the area S, and as their working load ranges from 200 to 1,000 lbs. per square inch, the factor of safety against sinking is from 3 to 10. The factor of safety against direct crushing of the timber should not be less than 10.

timber punch is interposed between the head of the ram and the pile. The best mode, however, of driving them, is by the aid of the screw, which will be mentioned in the next article.

403. **Screw Piles**, the invention of Mr. Alexander Mitchell, are piles which are screwed into the stratum in which they are to stand. The pile may be either of timber or iron, and that it may admit of being easily turned about its axis, should be cylindrical, or at all events octagonal. The screw blade, which is fixed on at the foot of the pile, is usually of cast iron, and seldom makes more than a single turn. Its diameter is from twice to eight times that of the shaft of the pile, and its pitch from one half to one-fourth of its diameter. The best mode of driving screw piles is to apply the power of men or of animals, walking on a temporary platform, directly to levers radiating from the heads of the piles.

As an example may be cited the cast iron piles already mentioned in Article 381, p. 572, as being used in the piers of railway bridges in India. Each of these was screwed into the ground by means of four levers, each 40 feet long, and each having eight bullocks yoked to it. According to this example, the greatest working load upon each screw of 4 feet 6 inches in diameter, *exclusive* of the earth and water above it, is nearly as follows:—

Pier 25 tons + superstructure 12 + train 30 = 67 tons = 150,080 lbs., being at the rate of nearly
100 lbs. per square inch of the horizontal projection of the screw-blade.

As these piles are screwed from 20 to 45 feet into the earth, the weight of earth above each screw-blade may be taken as ranging from 14 lbs. to 31 lbs. per square inch; so that the load on each screw blade, exclusive of the weight of earth above it, ranges from 3 times to 7 times that weight, and including the weight of earth, from 4 times to 8 times; results which correspond with the theory of Article 237, p. 379, if the angles of repose of the earth be assumed to range from about 28° to about 19°. (See p. 618.)

For the resistance of screw piles to wrenching, see page 502. From experiments by Mr. John Wood, C.E., it appears that the factor of safety, 6, is barely enough for cast iron screw piles, the greatest safe working stress being little more than 4,000 lbs. per square inch.

404. **Sheet Piles** are flat piles, which, being driven successively edge to edge, form a vertical or nearly vertical sheet, for the purpose of preventing the materials of a foundation from spreading, or of guarding them against the undermining action of water. They may be made either of timber or of iron.

Timber sheet piles are planks having a projection or feather along one edge, and a corresponding groove along the opposite edge.

They are of any breadth that can readily be procured, and from 2½ to 10 inches thick, and are sharpened at the lower end to an edge, which, in stony ground, may be shod with sheet-iron.

When a space is to be enclosed with sheet-piling, a range of *guide-piles* is first driven, being long rectangular piles at regular intervals apart of from 6 to 10 feet: these are driven to the same depth as bearing-piles. To the opposite sides of these, near the top, are notched or bolted a pair of parallel *string-pieces* or *wales:* these are horizontal beams, from 5 to 10 inches square, notched on the guide-piles to such a depth as leave a space between them of a width equal to the thickness of the sheet-piles. If the sheet-piles are to stand more than 8 or 10 feet above the ground, a second pair of wales is required near the level of the ground.

The sheet-piles are driven between the wales to about half the depth of the guide-piles, beginning with the sheet-piles next the guide-piles, and working towards the middle of each space between a pair of guide piles; so that the last or central sheet-pile acts as a wedge to tighten the whole.

In *iron sheet-piling* the guide-piles may be either tubular, or of a form of section like a trough-girder set on end (fig. 240, p. 524). The sheet-piles are also like trough-girders set on end, being plates stiffened by vertical ribs on the inner side. Their side edges are so formed as to make over-lapping joints, and their lower edges are wedge-shaped.

For example, in the foundations of Chelsea Bridge, the cast iron guide-piles are tubular, flat on the outer side, semi-cylindrical on the inner side, 12 inches in external diameter, and 1 inch thick, and are 27 feet long; the sheet-piles are cast iron plates, 10 feet long, from 6 to 7 feet broad, and 1 inch thick, stiffened by vertical ribs, which are from 4 to 6 inches deep, and from 10 to 20 inches apart, and by one horizontal rib of about the same dimensions at the upper edge.

405. **Timber and Iron-Cased Concrete Foundations.**—In foundations of this class the building rests on a mass of concrete (as to the strength and dimensions of which, see Article 239, pp. 381, 382), that mass being cased with sheet-piling of timber or iron, such as that described in the preceding article.

The sheet-pile casing is constructed first, and is sufficiently braced, transversely and diagonally, to enable it to resist the pressure to which it may be exposed, whether of water and mud from without, or of concrete, while in the soft state, from within. The soft material within the casing is then scooped out.

The concrete should be that described in Art. 230, p. 374, as *strong hydraulic concrete*, or "*beton*," and should be laid in layers of about a foot thick, each layer being either well rammed or thrown

in from a stage at least 10 feet high. Time should be given for the concrete to become firm before a heavy load is placed on it; for it has been shown by recent observations that intense pressure retards the setting of concrete.

The casing, besides facilitating the excavation of a bed for the concrete, serves to protect it afterwards from injury by such causes as the wearing action of a river current. When the casing is of iron, it is capable of bearing also a share of the load.

Sometimes a timber or iron-cased mass of concrete is combined with a system of bearing-piles, as described in Article 402, pp. 602 to 604.

406. **Iron Tubular Foundations** consist of large hollow vertical cast iron cylinders, filled with rubble masonry or concrete, such as have already been partly described in Article 381, pp. 572, 573.

The general construction of such cylinders and the mode of sinking them are shown by the vertical section, fig. 274. Amongst the auxiliary structures and machinery not shown in the figure are, a temporary timber stage from which the pieces of the cylinder can be lowered, and on which the excavated material can be carried away; and a steam engine to work a pump for compressing air.

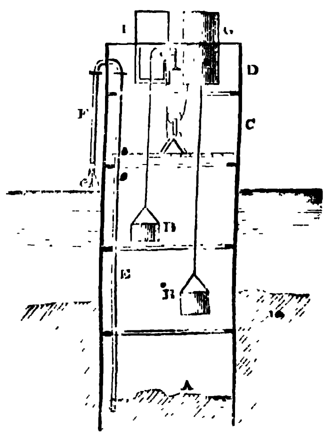


Fig. 274.

The following were the dimensions of the engine and pump used to supply air to a cylinder of the Theiss bridge formerly referred to; the diameter being 9·84 feet, and the greatest depth below the surface of the water to which the cylinder was sunk, 66 feet, corresponding to an absolute pressure of 3 atmospheres (or 2 atmospheres, that is to say, 29·4 lbs. on the square inch, above the ordinary atmospheric pressure). This is said to be the greatest pressure under which the excavators can work without danger to their health. (See p. 801.)

Diameter of steam cylinder (high pressure),......	8·67 inches.
,, of air-cylinder,............................	11·82 ,,
Length of stroke (the same in both cylinders),...	0·656 foot.
Number of revolutions per minute,..........from	100 to 120.
Horse power,....................................from	10 to 12.

From these data it appears that the volume of air supplied, measured at the ordinary atmospheric pressure, was from 100 to 120 cubic feet per minute. It appears that the number of persons within the cylinder at one time was from six to eight.

The cylinder consists of lengths of about 9 feet, united by internal flanges and bolts. The joints are cemented and made air-tight with a well-known composition, consisting of

Iron turnings,..............................	1,000	parts	by weight.
Sal-ammoniac,..............................	10	,,	,,
Flour of sulphur,...........................	2	,,	,,
Water enough to dissolve the sal-ammoniac.			

In some examples each joint is made tight by means of a ring-shaped cord of vulcanized indian rubber, lodged in a pair of grooves on the faces of the flanges.

The lowest length, A, of the cylinder, has its lower edge sharpened, that it may sink the more readily into the ground. The intermediate lengths, B, B, and the uppermost length, C, have flanges at both edges, upper and lower. The portion D, at the top, forms the "bell." The lower edge of the bell has an internal flange by which it is bolted to the cylinder below; its upper end is closed, and may be either dome-shaped, or flat, and strengthened against the pressure of the air within by transverse ribs, as in the figure. In the example shown the bell is made of wrought iron boiler plates.

E is a siphon, 2 or 3 inches in diameter, through which the water is discharged by the pressure of the compressed air.

F and G are two cast iron boxes, called "air-locks," by means of which men and materials pass in and out. Each of them has at the top a trap door, or lid opening downwards from the external air, and at one side, a door opening towards the interior of the bell, and is provided with stop cocks communicating with the external air and with the interior of the bell respectively, which can be opened and closed by persons either within the bell, within the box, or outside of both. These may be called the escape cock and the supply cock.

The bell is provided with a supply pipe and valve for introducing compressed air, a safety valve, a pressure gauge, and a large escape valve for discharging the compressed air suddenly when required.

At the lower flange of the division C is a timber platform, on which stands a windlass.

The apparatus is represented as working in a stratum of earth or mud, covered with water.

The operation of sinking a cylinder is analogous to that of sink-

ing a shaft with a drum-curb. (Article 391, p. 592.) The first operation is to lower the lowest length A of the cylinder, till it rests on the earth, with as many intermediate lengths B as are sufficient to reach a foot or two above the top water-level, and one additional length C, all bolted together. Then the bell is bolted on. The whole cylinder sinks to a depth depending on the material on which it rests. The engine then forces in air until the water is expelled from the cylinder. Workmen, with tools and buckets, can now pass in and out through the boxes or air-locks. To pass in, the operation is as follows:—Shut the supply-cock of the box, if not shut already; open the escape-cock—should there be compressed air in the box, it will be discharged; open the trap-door and enter the box; shut the trap-door and fasten it; shut the escape-cock; open the supply-cock; in a few minutes the box will be filled with compressed air at the same pressure with that in the bell; open the side door and pass into the bell. To pass out, the operation is as follows:—Should the escape-cock of the box be open, shut it; should the supply-cock be shut, open it; the box will soon be filled with compressed air, if not full already; open the side-door. enter the box, close the side-door, shut the supply-cock, open the escape-cock; when the air has fallen to the external pressure, open the trap-door and pass out. Some of the workmen (generally two) descend by a ladder or a bucket to the bottom of the cylinder, dig away the earth from its interior, and put it into buckets, which are raised by a set of men working the internal windlass, and sent through the air-locks; whence they are removed by an external windlass, not shown in the figure.

So soon as the excavation has been carried down to the level of the lower edge of the cylinder, the miners carry their tools and the lower division of the siphon E up to the platform; the whole of the workmen leave the bell; the great supply valve is shut, and the great escape-valve opened, so that the whole of the compressed air escapes. The cylinder being deprived of the support arising from the pressure of the compressed air against the top of the bell, sinks to a depth usually varying from one to two yards. When it has given over sinking, the great escape-valve is shut, and the great supply-valve opened, and the operation goes on as before, until it becomes necessary to put on an additional length of cylinder. This is done, while the pressure within and without are equal, by unbolting and taking off the bell, putting on a new length of cylinder on the top of C, which now becomes an intermediate length, removing the platform and windlass up to the new length, putting an additional length into the siphon, and replacing and bolting on the bell.

In the case taken as an example, each cylinder was sunk by

gangs of nine men working six hours at a time; and the earth (sand and clay) was removed at the rate of 15 buckets, each containing ·09 of a cubic yard, per hour; that is,

$$15 \times \cdot 09 = 1 \cdot 35 \text{ cubic yard per hour, by nine men;}$$
$$\text{or } \cdot 15 \text{ cubic yard per man per hour;}$$
$$\text{or } 6\tfrac{2}{3} \text{ hours of one man per cubic yard.}$$

The total volume of earth which has to be removed ranges, according to the stiffness of the material, from *once* to *three times* the volume formed by multiplying together the sectional area of the cylinder and the depth to which it is sunk. (See pp. 791, 801, 806.)

Care must be taken to keep the cylinder upright as it descends, by means of stays.

When the sinking of the cylinder has been completed, it is filled with masonry, or with hydraulic concrete; as to which, see Article 405, p. 606. About one half of the building is performed in the compressed air; the remainder, with the cylinder open at the top, the bell being removed.

Care should be taken to pack the concrete or masonry well below, and to bed it firmly above, each of the pairs of internal flanges.

In very soft materials it is sometimes necessary to drive a set of bearing piles in the interior of each cylinder, in order to support the concrete and masonry.

The earliest mode of sinking iron tubular foundations was that invented by Dr. Potts, in which the air is *exhausted* by a pump from the interior of the tube, which is forced down by the pressure of the atmosphere on its closed top. This method is well suited for sinking tubes in soft materials that are free from obstacles which the edge of the tube cannot cut through or force aside, such as large stones, roots, pieces of timber, &c.*

407. **Foundations made by Well-sinking** are in some respects analogous to iron tubular foundations. They are suitable for a soft and wet stratum with a firm stratum below it. They are made by sinking a sufficient number of cylindrical stone or brick-lined shafts, each on a drum-curb (see Article 391, p. 592), through the soft stratum, until the firm stratum is reached. These shafts are

* The method of sinking cylinders by the aid of compressed air was invented about 1841 by M. Triger. It was first used on a great scale a few years afterwards, by Mr. Hughes, at the bridge over the Medway at Rochester, executed from the designs of Sir William Cubitt, by Messrs. Fox, Henderson, & Co.

It was at first intended that the tubes should be sunk by the exhaustive process; but the remains of an old timber bridge, imbedded in the mud at the bottom of the river, rendered that impracticable; and the compressive process was then introduced.

then filled with rubble masonry, or with brickwork, so that each of them becomes a solid cylindrical pillar.

408. **Caissons.**—A caisson is a sort of flat-bottomed boat in which the foundation-courses and lower part of some structure which is to stand in water, such as a bridge-pier, are built, and floated to their intended site. The bottom of the caisson is a horizontal timber platform, fitted to form a permanent part of the foundation, as described in Article 399, p. 601. The sides are vertical, and are capable of being detached from the bottom. A seat is prepared for the platform, by excavation alone, by laying a bed of concrete, by driving a set of piles, or otherwise, as the occasion may require. The caisson is moored over that seat, and when the building within it has been carried to a sufficient height, it is gradually sunk, by slowly admitting the water, until the platform rests on its bed. The sides are then detached and removed.

The usual method of connecting the sides with the bottom is as follows:—The main supports of the bottom consist of a number of parallel transverse beams whose ends project beyond the sides; across the upper edges of the sides are laid an equal number of similar beams, into which the uppermost wales or longitudinal pieces of the sides are so notched as to be kept by the beams from being forced together by the pressure of the water. The projecting ends of the upper set of beams are connected with those of the lower set by long vertical iron bolts, outside the caisson, having a hook and eye joint a little above the lower beams; and by unfastening these the sides are at once detached from the bottom.

The dimensions of the timber used in the bottom are usually about the same as for a foundation-platform (Article 399, p. 601); those of the framework of the sides may be computed according to the principles of the strength of materials, so as to bear safely the greatest pressure of the water.

In an example described by Becker, used in building a bridge pier, the caisson was about 63 feet long, 21 feet broad, and 15 feet deep over all, the masonry within being about 18 feet broad. The cross beams were 10 inches square, and about 2 feet 10 inches apart from centre to centre; the upright standards of the sides were 10 inches square, and 5 feet 8 inches from centre to centre.

In some cases caissons have been built of bricks and cement in a graving-dock, coated with coal tar, and floated to the site of the work of which they are to form part: they are then sunk, and filled with concrete. (See p. 801, 810.)

409. **Dams for Foundations** are made for the purpose of excluding water from a space in which a foundation or some such structure is to be made. The materials principally used in them are timber, iron, and clay puddle, as to which last, see Article 206,

p. 344. Hydraulic concrete also is occasionally used, as to which, see Article 230, p. 374.

I. *Clay Dams.*—In still water of a depth not exceeding 3 or 4 feet, and on moderately firm ground, a clay puddle embankment forms a sufficient dam; care being taken, before commencing it, to dig a trench for its foundation, so as to remove loose and porous material from the surface of the ground.

II. *Coffer Dams.*—In greater depths, the essential part of an ordinary dam consists of two parallel rows of main piles and sheet piles (see Article 404, p. 605), enclosing between them a vertical wall of clay puddle. The upper wales of the two rows of piles are tied together by cross beams, which support a stage of planking for the use of the workmen. The main piles in one row are from 4 to 5 feet apart. The ground is excavated between the rows of sheet piles until a sufficiently firm bottom is reached, and the puddle rammed in layers.

The common rule for the thickness of a coffer dam is to make it equal to the height above ground, if the height does not exceed ten feet; and for greater heights, to add to ten feet one-third of the excess of the height above ten feet.

When the height exceeds twelve or fifteen feet, or thereabouts, three, and sometimes four or more, parallel rows of sheet piling are driven, thus dividing the thickness of the dam into two, three, or more equal divisions, each of six feet thick, or thereabouts; the outermost division is made of the full height, and the heights of the inner divisions are made less, so as to form a series of steps.

It appears from experience that a thickness of from two to five feet of clay puddle is sufficient to make a coffer dam water-tight; the additional thickness given by the rules above mentioned is required for stability, and the more so that the timber framework cannot be stiffened inside by diagonal braces between the rows of girder piles; for such braces would conduct streams of water along their sides through the puddle.

Another mode of obtaining stability is to make the dam simply of sufficient thickness to exclude the water, and to support it from within against the pressure of the water by means of sloping struts, abutting at their upper ends against the main piles of the inner face of the dam, and at their lower ends, in soft ground, against piles driven for that purpose, and in hard ground, against foot-blocks.

Let b be the breadth, in feet, of the division of the dam sustained by one such strut.

x, the depth of water,

w, the weight of a cubic foot of water, being 62·4 lbs. for fresh, and 64 lbs. for salt water.

Then, by the principles of Article 107, p. 166, equation 9, the total pressure of the water against that division of the dam is

$$P = w\, b\, x^2 \div 2; \text{..............................} (1.)$$

and the moment of that pressure, relatively to a horizontal axis at the level of the ground is

$$M = w\, b\, x^3 \div 6. \text{..............................} (2.)$$

Let h be the height above the ground at which the strut abuts against the dam, and i its inclination to the horizon; the thrust along the strut is

$$T = M \sec i \div h; \text{..............................} (3.)$$

and the scantling required to bear that thrust safely may be computed by the principles of Article 158, p. 238, equations 6, 7, 8.

When a coffer dam is to be exposed to waves, add together the greatest depth of still water in front of it, and *twice* the greatest height to which the crest of a wave rises above the level of still water, and put the sum for the greatest depth to which the dam is to be adapted (x in the formulæ). In shallow water on exposed parts of the coast, this amounts very nearly to making x equal to double the greatest depth of still water.

In firm ground impervious to water, planks laid horizontally on edge between a double row of guide piles may be substituted for sheet piling. The least thickness suitable for such planks is about $2\frac{1}{2}$ inches; and with guide piles five feet apart this is sufficient for a depth of about six feet; for greater depths, the thickness must increase in proportion to the square root of the depth.

For a rocky bottom, the following construction has been used by Mr. David Stevenson (see *Trans. Inst. of Civil Engineers*, vol. III.; also *Encyc. Brit.*, Article "Inland Navigation"):—Two parallel rows of vertical iron rods, three feet apart, were jumped into the rock to a depth of fifteen inches, to answer instead of guide piles; inside these rods, and supported by them, were two vertical linings of planks laid on edge horizontally, between which clay puddle was rammed; outside the iron rods were horizontal timber wales five feet apart vertically, or thereabouts; these were bolted together in pairs, through the dam, to which stability was given by means of inclined timber struts, as already described.

III. *Caisson Dams.*—Another mode of constructing a dam on a rocky bottom is to use a number of caissons, or flat-bottomed boats, suitably formed, so as to enclose the space which is to be guarded by the dam; when these have been floated to their proper places and moored, they are to be gradually sunk until they begin to rest on

the bottom; two rows of main piles, running respectively along the outer and inner faces of the enclosure of caissons, are now to be lowered vertically side by side until their lower ends rest firmly on the bottom, and bolted in that position to the sides of the caissons; the loading of the caissons, by means of stones or other heavy materials, and by admitting water, is now to be proceeded with, until either the whole or a considerable part of their weight rests on the main piles. A framework is thus formed, resting on the bottom by means of the main piles. A third row of piles, or posts, suitably framed to the inner row of main piles, is now to be set up parallel to and within them; and between these two rows, the dam, properly speaking, is to be formed in the manner already described, with two linings of planks and a puddle wall. When the dam is removed, because of the foundation or other work within it being finished, or because the work is to be interrupted, the caissons are to be unloaded and pumped dry, and floated away, so as to be available at a future time for the resumption of the same work, or the execution of another, as the case may be.

As to dams of this class, see Mr. Hodges's account of the Victoria Bridge.

Caissons, or boats, capable of being floated and grounded at will, as above described, are suitable where it is necessary, not to make a water-tight dam, but merely to obtain protection from a current that would otherwise impede or injure the work. (See Stevenson *On American Engineering*, Chapter VIII.)

IV. *Crib-Work Dams* are used where timber is abundant and cheap. Crib-work consists of a series of layers of logs, laid alternately lengthwise and crosswise, notched and pinned to each other at their intersections: the distance apart of the logs in each layer is three or four times their diameter. A skeleton frame of any required dimensions having been formed in this manner, is floated to its intended site, and there loaded with stones laid upon platforms supported by some of the upper layers of logs, until it sinks. It can then be used in the same manner and for the same purposes as the caisson dams of Division III. (On the subject of crib-work, see Stevenson *On American Engineering*; Hodges on the *Victoria Bridge*.)

V. *Wicker-Work Dams* will be mentioned further on.

410. **Excavating under Water, Dredging, and Blasting.**—Processes have already been described by which excavations are made under the water-level by the aid of some apparatus for excluding the water from the site of the excavation, such as iron cylinders filled with compressed air (Article 406, p. 607), or coffer dams (Article 409, p. 612). The present article relates to the making of such excavations by tools or mechanism, without excluding the

water. Cases in which the currents of the water itself are made available for that purpose will be considered in a later chapter.

I. *Protection of the Excavation.*—When an excavation is made under water in order to deepen a channel, it seldom requires to be protected; but when it is made with a view to the construction of a foundation, and there are loose materials, either in the ground excavated, or suspended in the water, it must be guarded against currents in the water, which otherwise would sweep those materials into it and fill it up. This may be done by caissons (Article 407, Division III., p. 613), cribs (Article 409, Division IV., p. 614), or by an enclosure of sheet piling, whether timber or iron (Article 404, p. 605); and if the excavation is for the purpose of making a piled or concrete foundation, the sheet piling may afterwards form the permanent casing of that foundation. (Article 405, p. 606.)

II. *Dredging by Hand* is performed by means of an implement called a "spoon," or "spoon and bag." It consists of a pole, at one end of which is fastened an iron ring, steeled at the forward edge, and forming the mouth of a bag of strong leather or coarse canvas. The ring is hung by a rope tackle capable of being wound up by means of a crab, and the further end of the pole is held by a man. As the rope is wound up the spoon is dragged forward along the bottom, against which the man who holds the pole causes the edge of the ring to press, scooping earth into the bag, until it arrives directly below the crab, when it is hauled up and emptied into a punt or mud barge.

In small depths of water, such as four or five feet, the labour and cost of this operation are not much greater than those of excavating similar materials on dry land. In greater depths the operation becomes more laborious and costly, nearly in proportion to the depth; and in depths of more than ten feet it is not applicable.

Another kind of hand dredge has a sort of sheet iron scoop instead of the ring and bag, and is suitable for rough and stony materials.

III. The *Dredging Machine* consists essentially of a pair of parallel chains, driven by pullies so as to move up the upper side and down the under side of an inclined plane, and carrying in soft ground a series of buckets, and in stiff ground buckets and rakes alternately; the rakes to break up the ground and the buckets to lift it. The upper end of the inclined plane is hinged, so that the lower end adapts itself to the level of the bottom. The machine works in a well in the middle of the after part of a strong barge, over the stern of which the buckets empty themselves into a punt or mud boat. The ordinary prime mover is a steam engine; but small dredging machines are also used, which are worked by hand. According to Mr. David Stevenson, a steam dredge of sixteen

horse-power will, under favourable circumstances, raise about 140 tons of stuff per hour (that is, about 100 or 110 cubic yards); and the cost ranges from an amount nearly equal to that of excavation in similar material on land (say about 8d. per cubic yard for sand and gravel) to about half that amount (or nearly 4d.). In general, the larger and more powerful the machine, the less is the cost of dredging. (See pp. 796, 797.)

IV. *Blasting Rock* in shallow water is nearly similar to the same operation on land, as to which see Article 207, p. 344. In general, proportionately more powder must be used than on land; for under water, it is desirable to shiver the rock into pieces that can be removed by dredging. In a good example of such operations, described by Edwards in the *Proceedings of the Inst. of Civil Engineers*, the weight of rock loosened was about between 5,000 and 6,000 times that of the powder exploded. (See p. 348.)

In deep water, the diving bell must be used in preparing the blasts. (See Appendix.)

V. *Removing Large Stones.*—Boulders and blocks of stone which are too large to be lifted by the dredging machine may either be split or blasted into smaller pieces, or may be attached, with the aid of diving apparatus, by means of a lewis (Article 251, p. 391), to a boat, and so lifted and carried away.

411. **Diving Apparatus** (I.) *for a single diver* consists essentially of a metallic *helmet*, usually spherical, and made of copper, enclosing the diver's head and resting on his shoulders, connected at its base with an air and water-tight dress, provided with a long flexible tube and valve, opening inwards, for supplying air from a compressing pump above water, an escape valve for foul air, opening outwards, about the level of the diver's chest, and some glazed openings (usually three in number), at the level of his eyes. Each of these openings should be furnished with a water-tight valve, which the diver can instantly close in the event of the glass being broken. The air-feed-pipe enters at the back of the helmet, and the air is conducted thence by arched passages over the diver's head to points near the glazed eye-holes. By this arrangement the entrance of water is prevented, in the event of the feed-pipe bursting. To overcome the buoyancy of the apparatus, and enable the diver to sink, his waterproof dress is loaded with about a hundredweight of lead, part in the soles of the shoes, part fastened to the breast and back. He usually hauls himself up by means of a rope; but should he wish to ascend suddenly he has only to close the escape-valve, when the air inflates the waterproof dress and causes him to float to the surface The helmet is supplied with a speaking apparatus and an electric lamp. Telephones are also fitted for communication with assistant above; the chimney has

a flexible discharge-pipe ascending to the surface, with a valve opening outwards. This lamp is required more especially in turbid water. In America a diving helmet has been used made wholly of glass.

II. The *Diving Bell* commonly used is shaped like a rectangular box with rounded corners, measuring about six feet by four feet horizontally, and five feet high, two inches thick in the top and upper part of the sides, and increasing to three and a-half inches or thereabouts at the lower edge, for the sake of stability. It usually weighs about five tons, and displaces three and a half tons of water, or thereabouts, when quite filled with air: the difference is the load on the crane and windlass by which it is lowered and raised. It has a number, not usually exceeding twelve, of bull's eyes, or glazed holes in the top to admit light; they are eight or ten inches in diameter, and the glass about two inches thick. The flexible feed-pipe for supplying compressed air is about three inches in diameter. If the quantity of air required be calculated according to the data already stated as to the supply of foundation-cylinders (Article 406, p. 608), or according to the usual practice in public buildings, it should amount to about twelve cubic feet, measured at atmospheric pressure, *per man per minute.* Signals may be made by persons in the bell to those at the pumps and crane by pulling cords and ringing bells.

III. The *Diving Boat* (of which different kinds have been invented by Dr. Payerne and others) is a diving bell on a large scale, conveniently shaped for being moved about, and provided with a magazine of compressed air, contained in a casing surrounding the working chamber or bell. This magazine answers the purpose of the air-bladder of a fish, by enabling those within the bell to make it sink and rise at will; for by injecting water with a forcing-pump into the magazine, the boat becomes heavier, and sinks; and by opening an escape-cock at the bottom of the magazine, the water is forced out by the compressed air, and the boat becomes lighter and rises.

412. **Embanking and Building under Water.** (See also Article 205, p. 344.)—Embankments under water may be made by tipping in the material from a stage supported on posts or on screw piles, or from boats; a moveable inclined plane or shoot being used to direct the material to the spot where it is to fall. Stones and gravel are in general the only materials whose stability can be relied on when exposed to currents in the water; and the diameter of the smallest pieces should not be less than about one twenty-fourth part of the velocity of the current in feet per second. When the outside of an embankment is formed with stones, the interior may be filled with smaller and softer materials. In water not

agitated by waves an embankment of loose stones will stand at a slope ranging from that of 1 to 1 to that of 2 to 1; but where it is exposed to waves, it must be faced with blocks set by hand, with the aid of diving apparatus, if necessary, the least dimension of any block in the facing being not less than two-thirds of the greatest height of a wave from trough to crest. Further remarks on this will be made in a later chapter.

A loose stone embankment may be protected against waves and currents by means of wooden crib-work.

Hydraulic concrete can be laid under water simply by pouring it into an excavation, or into a space enclosed with a timber or iron casing, the surface of each layer, in deep water, being levelled and smoothed with the aid of diving apparatus. Regular masonry, whether consisting of stones, or of large blocks of hardened concrete, requires the aid of diving apparatus during the whole process of building. (See Art. 230, p. 374; also p. 436.)

For the facing of sea-works exposed to the action of waves in deep water, such as breakwaters, enormous blocks of hydraulic concrete are sometimes used, measuring from 12 to 27 cubic yards in volume. For the protection of these against the corroding action of sea-water, a method has been introduced of coating them all over, to a thickness of about three inches, with asphaltic concrete, composed of two parts of asphaltic mastic (Article 234, p. 376) and three of broken stone. (See a paper by M. Léon Malo, in the *Annales des Ponts et Chaussées*, 1861; also Appendix.)

In ashlar masonry which is to be exposed to very violent shocks from the waves, such as that of lighthouses, the stones, besides being fastened together by metal cramps, are sometimes bonded also by dove-tailing, in the manner shown in plan by fig. 275, which represents part of a course of a lighthouse. This was first practised by Smeaton at the Eddystone lighthouse. Its chief use is to resist the tendency which the stones at the face of a wall have to *jump out* immediately after receiving the blow of a wave. Stones of different courses are sometimes connected by *tabling*, which consists in making flat projections on the beds of the stones which fit into corresponding recesses in the beds of those above and below them.

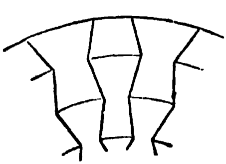


Fig. 275.

ADDENDUM *to Article* 403, p. 605.—DISC PILES (the invention of Mr. Brunlees) have a disc at the foot, and are lowered by driving the sand from below the disc by means of a stream of water.

PART III.

OF COMBINED STRUCTURES.

CHAPTER I.

OF LINES OF LAND-CARRIAGE.

SECTION I.—*Of Lines of Land-Carriage in General.*

413. **General Nature of Works.**—The works which constitute a line of land-carriage (exclusive of the buildings and machinery by the aid of which the traffic is carried on) may be divided into PERMANENT WAY and FORMATION; the permanent way being that part of the structure which directly bears the traffic, and the formation, the whole of the rest of the works, whose object is to make and preserve a suitable passage for the permanent way across the country. In a restricted sense, the word *formation* or *forming* is applied to the base or surface on which the permanent way directly rests.

As the methods of constructing the works which constitute the FORMATION, in the widest sense, have been described in the preceding part of this treatise, it is only necessary in the present chapter to enumerate them (referring to the places where they are described in detail), and to state the principles according to which they are adapted to particular lines of conveyance. They may be thus classed:—

I. *Earthwork*, consisting of cuttings and embankments, to make passages through hills and over valleys respectively. (See Part II., Chapter II., p. 315.)

II. *Fences.*—As to temporary fences, see Article 189, p. 333. Permanent fences will be again referred to.

III. *Drains*, which are treated of in the same chapter in their relation to earthwork. As to the masonry of large drains, see Article 297 A, p. 433.

IV. *Retaining Walls.*—(See Articles 265 to 275, pp. 401 to 413.)

V. *Level Crossings* of other lines of communication will be again mentioned further on.

VI. *Bridges*, which may be classed according to their purposes, or according to their materials.

The purpose of a bridge may be—

A. To cross over or under some existing line of communication, which it is either impracticable or inexpedient to cross on the level. In this case there are in general certain minimum dimensions fixed by law or by agreement for the passage to be allowed for the existing line, which will be again referred to.

B. To cross a valley, in which an embankment would be impracticable, or too expensive, or otherwise inexpedient. In this case the bridge is called a *viaduct*.

C. To cross a stream, river, estuary, strait, or other piece of water. The principles to be observed in this case, in order that the current may not be impeded, nor the navigation (if any) injured, will be referred to in a subsequent chapter.

The materials of a bridge may be—

a. Masonry or brickwork; as to which, see Part II., Chapter III., p. 349, and in particular, Section VIII., p. 413.

b. Timber; as to which, see Part II., Chapter IV., p. 437, and in particular, Article 336, p. 465, and Articles 341 to 349, pp. 475 to 492.

c. Iron; as to which, see Part II., Chapter V., p. 494.

As to the ordinary *foundations* of bridges, see Part II., Chapter III., Section IV., p. 377; and as to the more difficult kinds of foundations, see Chapter VI., Section II., p. 601.

VII. *Tunnels;* as to which, see Part II., Chapter VI., Section I., p. 588.

The PERMANENT WAY of a line of land-carriage is either a *road*, a *railway*, or a *tramway*; the essential distinctions being that a *road* presents a firm surface of a certain breadth, which can be traversed by vehicles over all its parts and in all directions; a *railway* confines vehicles to certain definite tracks, by means of rails on which specially formed wheels run; a *tramway* is intermediate between these, and consists of flat rails laid on a part of the surface of a road, and so formed that vehicles with wheels suited for an ordinary road can run upon them when required.

414. **Selection of Line and Levels.**—The selection of the position of a line of conveyance depends on statistical and commercial, as well as mechanical considerations, but although the former come frequently under the notice of the engineer, they are foreign to the proper subject of this treatise.

In a purely engineering point of view, the object to be aimed at in laying out the course of a line of communication is to convey the traffic with the least expenditure of motive power consistent with due economy in the construction of the works. Economy of motive power is promoted by low summit-levels, flat "*gradients*" (as the rates of declivity of lines of land-carriage are called),

easy curves, and a direct line; but limitations to the height of summits, the steepness of gradients, and the sharpness of curves, limit also the power of adapting the line to the inequalities of the ground, and so economizing works.

The data required by the engineer in order to enable him to select a line, and the means of obtaining these data, have been stated in Part I., Chapter I., Article 11, p. 9, and further explained in subsequent articles of that part; and as regards borings, pits, and mines, in Part II., Article 187, p. 351, and Articles 391, 392, pp. 589 to 595. The general character of the inequalities of the ground, or "features of the country," and the modes of representing them, have been described in Part I., Articles 58, 59, 60. pp. 93 to 98.

A projected line of communication may either be limited to the connection of two points in the same valley, or it may have to connect points in two or more valleys, by crossing the ridges between them. In the former case, there is no summit-level to cross; in the latter, there may be one or more summit-levels. In general, the best point for crossing a ridge is the lowest *pass* (see Article 58, pp. 94, 95) which occurs in the district to be traversed; but cases may arise in which a higher pass is to be preferred to a lower, because of its being more easily accessible, or because of its offering greater facilities for cutting or tunnelling. The ridge ought to be crossed as nearly as possible at right angles.

When a line of communication has to cross a great valley, the following principles are to be observed as far as practicable:—To choose a narrow part of the valley; to cross the deepest part of it as nearly at right angles as possible; to find, if possible, firm ground for the foundation of a viaduct, or of a high embankment.

The principle of crossing obstacles as nearly as possible at right angles applies to bridges over rivers, and over or under other lines of communication. The cost of a skew bridge increases nearly as the square of the secant of the obliquity.

When a line of communication runs along one side of a valley, the obstacles which it has to cross are chiefly the small branch valleys that run into the main valley, and the promontories or ends of branch ridge-lines that jut out into the main valley between th. branch valleys. In this case the greatest economy of works is attained by taking a serpentine course, concave towards the main valley in crossing the branch valleys, and convex towards the main valley in going round the promontories, except where narrow necks in the promontories and narrow gorges in the branch valleys enable a more direct course to be taken with a moderate amount of work.

In ascending the head of a steep valley towards a high pass, it

may be sometimes necessary to take a serpentine or even a zigzag course in order to obtain a sufficiently easy gradient, independently of considerations of economy of work. In a few instances a projecting promontory or spur of a mountain has been made available for the ascent of a line of conveyance to a pass, by laying out the line in a spiral course, each coil of the spiral passing first through the promontory by a tunnel, and then winding round outside of it.

It is obviously difficult to lay out a line of conveyance so as at once to accommodate the traffic which passes along the lower part of a large and deep valley, and that which passes over the pass at its head; for in order to reach the summit easily, the line must quit the lower part of the valley at a certain distance from the pass, and ascend gradually along the sides of the hills, so that in some cases a branch line may be required for the lower part of the valley.

In the formation of all lines of conveyance, it is advisable to avoid long reaches of level line in cutting, as being difficult to drain. (See Article 192, p. 335.)

As to crossing a great plain, see Article 203, p. 342. In this case the level of the line of communication is generally fixed so as to be sufficiently high above the highest water-level of floods.

415. The **Ruling Gradient** of a line of communication means the steepest rate of inclination which prevails generally on the line; being exceeded only on exceptional portions, where auxiliary motive power can be provided, or where the loads to be conveyed up the ascent are lighter than on other portions of the line. The economy with which the works can be constructed depends mainly on the steepness admissible for the ruling gradient.

Two things are chiefly to be considered in fixing a ruling gradient: the motive power available in ascending, and the avoidance of waste of power in descending.

Let W denote the greatest gross load to be dragged up an ascent;
f, the proportion of the resistance to the load on a level;
i, the sine of the angle of inclination of the ascent; then

$$(f + i)\,W,$$

is the greatest resistance to be overcome in ascending the ruling gradient; and this should not exceed the greatest tractive force which the prime mover is capable of exerting. Let P be that force; then

$$\left.\begin{array}{l} (f + i)\,W \text{ should not be greater than } P\text{; or, in} \\ \quad\text{other words,} \\ i \text{ should not be greater than } \dfrac{P}{W} - f. \end{array}\right\} \ldots (1.)$$

The fulfilment of this condition is essential. Another condition, which it is desirable to fulfil, if possible, is, that no mechanical energy shall be wasted through the necessity of using brakes, or of backing the prime mover, in order to prevent excessive acceleration of speed in descending the ruling gradient; and to fulfil this condition

i should (if possible), not exceed f. ....(2.)

The co-efficient of resistance f differs very much for different sorts of permanent way. In each case it consists of two parts; one arising from friction, and constant at all speeds, and another arising from vibration, and increasing with the velocity; so that it may have different values in the formulæ 1 and 2; that in formula 1 corresponding to the *least speed* of ascent consistent with the convenience of the traffic, and that in formula 2 corresponding to the *greatest speed* of descent consistent with safety.

When the traffic is heavier in one direction than in another, the ruling gradient in the direction of the ascent of the lighter traffic may be the steeper.

As a general consequence of these principles, it is obvious that the less the proportion of the resistance on a level to the load, the flatter must be the ruling gradient, and the flatter the ruling gradient is the heavier are the works, and the more difficult is it to lay out the line. Such, for example, is the case with railways, as compared with roads. In railways additional expense and difficulty are occasioned by the necessity of certain limitations as to the sharpness of the curves; but these will be explained in Section IV.

Section II.—*Of Roads.*

416. **Resistance of Vehicles and Ruling Gradients.**—The vehicles capable of being used on roads may be distinguished into sledges and wheel-carriages. The only cases in which sledges are suitable vehicles for roads are those in which the surface is either too soft or too steep to admit of the use of wheel-carriages with safety. Their resistance on roads has not been determined precisely by experiment.*

The resistance of wheel-carriages on roads consists of a constant part, and a part increasing with the velocity. According to General Morin, its proportion to the gross load is given approximately by the following formula:—

$$f = \{a + b\,(v - 3{\cdot}28)\} \div r; \ldots\ldots\ldots\ldots(1.)$$

* The resistance of an iron-shod sledge on hardened snow is stated by Kossak to be about 1-30th of the gross load.

where r is the radius of the wheels *in inches*, v the velocity in feet per second, and a and b two constants, whose values for carriages with springs are as follows:—

	a	b	f for Wheels of 18 Inches Radius. $v=14\cdot67$	$v=7\cdot33$
For good broken stone roads,	·4 to ·55	·025	·038 to ·046	·028 to ·036
For pavements,....... from	·27	·068	·060	·030.
to	·39	·03	·041	·028.

For carriages without springs, the constant b is about $3\frac{1}{2}$ times greater than for those with springs.

The following table is founded chiefly on experiments by Sir John Macneill:—

	f
Stone pavement,.............................	1-68th = ·015
Broken stone road on a firm foundation,	1-49th = ·020
Broken stone road on a foundation of flints,..	1-34th = ·029
Gravel road,..	1-15th = ·067
Soft sandy and gravelly ground,...........	1-7th = ·143

Telford estimated the average resistance of carriages on a level part of a good broken stone road at *one-thirtieth* of the gross load; and according to the principle expressed in Article 415, equation 2, he assigned 1 in 30 as the ruling gradient which ought, *as far as possible*, to be adopted on a turnpike road.

If the tractive force which a horse can exert steadily and continuously at a walk be estimated at 120 lbs., the adoption of a ruling gradient of 1 in 30, the resistance on a level being 1-30th of the load, insures that each horse shall be able to draw up the steepest declivity of the road a gross load of

$$120 \times 30 \div 2 = 1{,}800 \text{ lbs.}$$

A horse can exert, for a short time, an effort two or three times greater than that which he can keep up steadily during his days' work; and thus steeper ascents for short distances may be surmounted.

In the roads laid out by Telford, the ruling gradient of one in 30 is adhered to, wherever it is practicable to do so; and sometimes considerable circuits are made for that purpose. Occasionally, however, he found it necessary to introduce steeper gradients for a short distance, such as 1 in 20, or 1 in 15.

417. **Laying out and Formation of Roads in General.**—Heavy

works of earth and masonry seldom occur on lines of road, which are often, throughout the greater part of their extent, made on the natural surface of the ground. In this case the operation of forming the road consists simply in digging, in ground that is level across, a drain at each side of the road, and in ground that has a sidelong slope, a drain at the uphill side; throwing the earth from the drains on the track of the intended road, so as to raise it slightly above the adjoining ground, and levelling any small inequalities that occur in its course. According to M'Adam,* this is all that is required preparatory to laying the covering or "metal" of the road, even in swampy ground. According to other authorities, it is advisable, in marshy ground, to prepare a foundation for the road by means resembling those employed in embanking over soft ground (Article 204, p. 342); for example, by digging a trench 2 or 3 feet deep, and filling it with clean sand or gravel, as a base for the road; or by spreading a layer of dried peat, or of fascines, so as to form a sort of raft to float on the morass. When *fascines* are used for this purpose, they will rapidly decay unless they are constantly wet. They consist of bundles of twigs, 20 feet long, or thereabouts, and from 9 to 12 inches in diameter, and are laid in layers alternately lengthwise and crosswise, and fastened with pegs, until a bed is formed about 18 inches deep, over which gravel is spread.

418. **Breadth and Cross-section.**—For the ordinary breadth of the carriageway of a turnpike or main road, about 30 feet is a sufficient width, with 5 or 6 feet additional for a footway at one side.

For cross-roads smaller widths are sufficient, such as 20 feet for the carriageway, and 5 feet for the footway.

The widths prescribed by law in Britain for those parts of public roads which are interfered with by railways are as follows:—

Turnpike roads,................................	35 feet.
Public carriage roads (not turnpike),......	25 ,,

For the widths of roadways in populous towns and their neighbourhood no general rule can be laid down.

In some of those cases in which the traffic is greatest, the width of carriageway is about 50 feet, with a pair of footways, each from 10 to 15 feet wide.

The carriageway should have a slight rise or convexity in the middle, in order that water may run off it towards the sides; and for that purpose from 4 inches to 6 inches is sufficient. This convexity should be given to the *formation*, so that the thickness

* See M'Adam *On Roads*, ninth edition, 1827.

of covering may be uniform. The footways should be nearly level with the highest part or crown of the carriageway, and have a very slight slope towards the carriageway.

For the form of cross-section of the convexity of the carriageway, Telford recommends a very flat ellipse; but Mr. Walker prefers two straight lines, connected by a short curve at the crown. He advises, also, that every part of a road should, as far as practicable, have a slight declivity longitudinally, to facilitate drainage.

419. **Drainage and Fencing.**—The side-drains, which have already been mentioned, are similar to those of pieces of earthwork, as to which, see Articles 190, 193, 202, pp. 334, 335, 342. A depth of 2 or 3 feet is in general sufficient for them; and when they are open ditches, they may be from 3 to 4 feet wide at the top. If covered, they may consist of earthenware tubes of 6 inches diameter, or thereabouts, on an average, or built culverts of about 12 inches square. Roadways in towns are in general drained into underground sewers.

The *gutters* or *channels* run along each side of the carriageway, and are usually about 3 inches deep. They collect the surface-water from the road, and discharge it into the side-drains through transverse tubes, which pass below the fences and footway.

Mitre drains are small underground tile drains or tubes, diverging obliquely from the centre line of the roadway at intervals of 60 yards or thereabouts, and leading, with a declivity of about 1 in 100, into the side-drains.

In towns the channels discharge their water into the sewer through passages called *gully-holes*, sometimes having horizontal openings covered by gratings, sometimes vertical openings in the curb of the footway. In order to prevent the escape of foul air through them, they are provided with siphon traps, or with valves opening inwards.

When a road is drained by an open ditch, the fence should be between the ditch and the road. The permanent fences of roads are usually either hedges or walls. According to Telford, they should not exceed 5 feet in height, in order that the sun and wind may have free access to the road to dry it.

420. **Broken Stone Roads.**—The true principles of the construction of roads covered with broken stone were discovered by John Loudon M'Adam, and are fully described in his work *On Roads* already referred to.

The stone, or "*road metal*," should be hard, tough, and durable. (On these points, see Part II., Section I., pp. 349 to 363.) The best materials are granite (p. 355) and trap-rock, or whinstone (p. 356). Hard compact limestone (p. 359) may also be used, and

gravel composed of flints (p. 357); but all flints should be broken into angular pieces, as if for making concrete.

The stones are broken down by means of a hammer with a steeled face, into smaller and smaller pieces, until at length they are reduced to pieces roughly approximating to a cubical shape, and not exceeding 6 ounces in weight, which, on an average, is the weight of a cube of stone of 1·6 inches in the side. M'Adam directed each road inspector to carry a small balance, so as to be able to test the weight of a few stones from each heap. The Stone-Breaking Machine (see Appendix) breaks stone into cubes of about $1\frac{1}{2}$ inch in the side, with an expenditure of power at the rate of from 1 H.P. to $1\frac{1}{2}$ H.P. for each cubic yard broken per hour.

Besides breaking all gravel into angular pieces, it should be screened, to clear it of earth.

The road metal, thus prepared, is to be evenly spread over the road with a shovel and rake, in three successive layers of between 3 and 4 inches deep, each layer being left to be partly consolidated by traffic before another is laid, or, still better, by the use of a steam roller, as to which see Appendix, p. 786; and thus is formed a firm, compact bed of angular fragments of stone about 10 inches thick, which is impervious to water, or nearly so, and which soon acquires a smooth surface.

According to M'Adam, 16 inches is the greatest thickness of metal required for any road, from 5 to 9 inches being often sufficient; and his practice was to lay the metal simply on the natural ground, with no preparation except levelling inequalities and digging drains, as described in Article 117, p. 625.

According to the practice of Telford, before laying down the metal, a foundation or "*bottoming*" is laid, consisting of pieces of durable, but not necessarily hard stone, measuring from 4 to 7 inches in each dimension. The largest of those pieces are set by hand, with their largest sides resting on the formation, and between these the smaller pieces are packed, so as to form a compact layer about 7 inches deep in the centre of the road, and 4 inches deep at the sides, part of the convexity being made in this manner. Above this bottoming the metal is spread as already described.

A broken stone road is repaired by thoroughly moistening it with water, then slightly loosening the upper surface with a pick, and spreading uniformly over it a layer of metal, which should be carefully rolled. Unless the surface is first loosened, the new metal will not *bind* or consolidate with the old. The practice of repairing roads by patches, called "darning," is bad.

In order to make the traffic on a broken stone road easier when it is first laid, a layer of sand and gravel, called "*blinding*," is sometimes spread over it; but this practice is a bad one, for the sand

and gravel work their way between the fragments of stone, and prevent their ever forming so compact a mass as they ought to do.

When mud forms on the surface of a road, it is to be removed by scraping; but if the road is well made of good materials, little of that work is required.

Wheels of small diameter are the most destructive to roads.

According to Telford, the load on a broken stone roadway ought not to exceed one ton on each wheel: the tire of the wheel, for a load of one ton, being four inches broad. The limitation as to load agrees with general practice; but the breadth of four inches for a load of one ton per wheel appears to be only necessary for vehicles without springs; for those properly provided with springs, a breadth of from 2 to $2\frac{1}{2}$ inches is sufficient under any load of ordinary occurrence, provided they are to run on firm and compact roadways only. On soft and loose roadways an additional breadth of wheel prevents the resistance from being so great as it would be with narrow wheels. The consolidation of a broken stone road may be hastened by rolling it with a cast iron roller weighing from 1 to 3 tons if drawn by horses, or by steam rollers. (See p. **791.**)

421. **Stone Pavements.**—The *foundation* of a stone pavement may consist either of a layer of hydraulic concrete, or of rubble masonry set in hydraulic mortar, from 6 to 9 inches deep;

Or of three successive layers of broken stone road metal, each about 4 inches deep, consolidated by allowing the traffic to run upon them for a time, or, still better, by rolling,

Or of three well-rammed layers of gravel, each 4 inches deep, with a layer of sand about 1 inch deep on the top..

The best materials for stone pavements are syenite and granite, the hardest that can be found; and the next, trap or "whinstone." Stones of a laminated structure are to be avoided if possible; and should it be absolutely necessary to use them, they are to be set with their beds or laminæ on edge.

Paving-stones should be roughly squared, special care being taken that they do not taper downwards. They are to be set in regular courses, running across the roadway, and breaking joint with each other. In order that the stones may not tend to cant or tilt over, their depth in a vertical direction should be somewhat more than double their horizontal breadth: for the same reason, the length should be equal to the depth, or not much greater The dimensions usually adopted are—

Breadth (in a direction along the roadway),......	4 inches.
Depth (in a vertical direction),......................	9 „
Length (in a direction across the roadway),...........................	from 9 to 12 „

Paving-stones are sometimes made to taper slightly *towards the top*, so that their joints are close below, and open to the extent of an inch or thereabouts above, the wedge-formed spaces thus left being filled with gravel, or chips of stone, imbedded in bituminous cement. (Article 234, p. 376.) This gives a more secure footing to horses than a close-jointed pavement.

Small pieces of granite, nearly cubical, and measuring about 4 inches each way, have been used at Euston Square Station. They rest on a layer of sand 1 inch deep, and three layers of gravel mixed with chalk, each 4 inches deep, and are set as close as possible.

Paving-stones are rammed into their places with a wooden rammer or beetle, weighing about 55 lbs. A small steam-hammer has been sometimes used for this purpose.

They may be covered, when first laid, with a blinding of sand and fine gravel, about an inch or an inch and a-half deep, to fill the joints by degrees.

Their joints may be made water-tight by being laid in cement or hydraulic mortar; or in iron-turnings, which rust, and make a sort of cement with the sand and gravel of the blinding that works its way into the joint; or in a bituminous cement, or by being *grouted* with hydraulic lime in a semi-fluid state after being laid.

Rubble or *Boulder Pavement* consists of stones of irregular shapes set in a bed of sand or gravel. It causes great resistance to vehicles, is liable to irregular sinking, and requires frequent repair.

The chief disadvantage attending the use of well-made stone pavement in towns is its liability to be disturbed for the purpose of laying gas and water-pipes and small sewers. One method of obviating this is to provide "*side-trenches*" to contain those under-ground works, being narrow excavations lined at the sides with brick walls, and situated under the outer edge of the foot-pavement, by the flags of which they are covered. The wall of the side-trench next the roadway is strengthened against the pressure of the earth by means of transverse walls, with openings in them for the passage of sewers and pipes; and between those transverse walls the longitudinal wall is slightly arched horizontally, like the retaining wall in fig. 176, p. 412. The other longitudinal wall of the side-trench forms the back of a row of cellars under the foot-pavement. The side walls of the cellars are in a line with the transverse walls of the side-trench, and act as buttresses to give it stability. The following dimensions are given from actual practice:—The side-trench is 13 feet deep from surface of footway to foundation, 2½ feet wide inside, and has cross walls at every 7 feet; the brickwork is one brick, or 9 inches thick. It contains an oval sewer-pipe of 27 × 18 inches, a 10 inch water-pipe, and a 10 inch

gas-pipe. Sewers which are large enough to be traversed by men may be repaired by getting access to them through subterranean passages leading into them from trap-doors in the foot-pavement.

Another method of obviating the necessity for raising the pavement of the carriageway is to have a "*sub-way*" or tunnel under the street, containing the sewer and the gas and water-pipes. This method has hitherto been tried for short distances only.

422. **Footways of Roads.**—In country roads the construction of the footways is the same with that of a broken stone road, except that smaller and less hard materials are used, and that from $2\frac{1}{2}$ to 4 inches is a sufficient thickness. The footway should have a declivity of about 2 inches towards the channel, its lowest edge being not more than 9 inches above the bottom of the channel, and its side towards the channel being formed either by a slope of from 1 to 1 to $1\frac{1}{2}$ to 1, or by a *curb-stone* set on edge, from 4 to 6 inches thick. To consolidate footways, a cast iron roller may be used, weighing from $\frac{1}{4}$ to $\frac{1}{2}$ a ton.

In streets the footways have a foundation of concrete, broken stone, gravel, or sand, and are covered with flagstones, usually from $1\frac{1}{2}$ to 4 inches thick, being thinnest for the strongest material. The best materials are those which are hardest, toughest, and least pervious to water, such as hornblende slate, the harder kinds of clay slate, gneiss, strong sandstone and compact limestone.

422 A. **Concrete Pavements** were introduced by Mr. Joseph Mitchell. They consist of broken stone road metal, well mixed with hydraulic mortar.

423. **Bituminous or Asphaltic Pavements** consist of a thin layer of what has been described in Article 234, p. 376, as "Bituminous Concrete," laid on a foundation of broken stone. The formation of the roadway has a convexity of 1-100th of the breadth.

The foundation consists of road metal, as described in Article 420, p. 626, laid in a layer of 4 inches deep for the carriageway, and 2 inches deep for the footway, and consolidated with a rammer of 55 or 56 lbs. weight, or with a cast iron roller.

The covering, which is about $1\frac{1}{2}$ inch thick for a carriageway, and $\frac{3}{4}$ inch thick for a footway, consists of a mixture of road metal or gravel and "bituminous mortar." The proportions of its ingredients have been given in Article 234, p. 376. The order in which they are to be combined is the following:—Having melted the bitumen, add the asphalt broken small, then the resin oil, then the sand, and lastly the broken stone. To test the composition, a specimen of it is cooled in water to the temperature of about 80°; a piece of plank, having two four-sided pyramidal points of iron on the under side, is laid with one point resting on a formerly-tried

standard sample, and the other on the new sample; a man stands on the middle of the plank, when the impressions on the standard sample and new sample should be of equal depth. That depth should be about 3-10ths of an inch for carriageways and 2-10ths for footways, the latter requiring the stronger material. Should it prove too hard, bitumen and resin oil are to be added; should it prove too soft, asphalt and sand. The covering is laid on the roadway in the hot state, in rectangular sections; its surface is sprinkled with sand, and the surplus sand swept off, and it is then left to cool. No artificial asphalt is equal to natural asphalt for making roads.

To make bituminous roadways cold, asphalt is to be broken as for road metal, spread about 2 inches deep, wet all over with coal-tar, and rammed with a 56 lb. beetle.

To repair the surface of a bituminous roadway, dissolve one part of bitumen in three of pitch oil or resin oil; spread 10 ounces of the mixed oil over each square yard of roadway, and sprinkle on it 2 lbs. of asphalt in powder; then sprinkle the surface with sand, and sweep away the loose sand.

Good bituminous pavements under constant traffic should wear at the rate of about 1-40th of an inch a-year.

424. **Plank Roads** are useful in newly settled countries in which timber is abundant.

425. **Wooden Pavements** have come into extensive use in the last five years. The principal advantages they offer over stone and M'Adam are the diminution in tractive force necessary, their noiselessness, and their giving a better foothold to horses. The disadvantages, on the other hand, are principally of a sanitary nature; as they absorb and retain noxious matter, the pressure of water required for cleansing them is much higher than for other forms of roads. The life of wood pavements is from nine to ten years, as nearly as may be judged from experience so far. There are several forms of wood pavement now in use, which differ in various particulars. Almost all agree in forming a concrete foundation of some kind or other, and making the superstructure watertight: if this can be satisfactorily attained, nothing more can be desired, but it is doubtful whether it is, and if water get between the joints of the blocks it naturally causes the wood to swell, and has in certain cases been known to tear up the kerb-stones. A method which has been employed extensively in America, is to lay the wood upon a foundation of sand, and to fix the blocks with wedges partly driven into the sand, the portion above the wedges being filled with concrete or cement. This method consolidates the sand whilst allowing of natural filtration. The method most in use in this country consists in forming a bed of concrete, directly upon which, in some cases, the wood blocks are laid, with fibres vertical.

sizes of 4″ × 3″ × 5″, the largest dimension being across the street and the next largest downwards, the spaces being filled with asphalt. In a second method, tarred felt is placed over the concrete and between the blocks. Whilst, in a third, the blocks are saturated with a liquid asphaltic mastic, the blocks being joined with the same material. The method which has received most favour, consist in laying 1 inch sand over the concrete, 1 inch boarding parallel to the street over this, the blocks being placed thereon, and the spaces of about ½ inch run in with asphalt and grouting. In every case the surface is spread over with sand or grit.

Section III.—*Of Tramways.*

426. **Stone Tramways** consist of a pair of parallel ranges of oblong blocks of granite, about 4½ feet apart from centre to centre, with their upper surfaces forming part of the surface of a road, each block being from 2 to 4 feet long, about 10 or 12 inches broad, and of the same depth with the rest of the covering of the roadway.

427. **Iron Tramways** are in fact railways, with the rails so formed that their upper surfaces form part of the surface of a road or street. According to the ordinary construction, the rails are of wrought iron or steel, in lengths of 24 or 25 feet, 4 inches broad, and weighing from 30 to 60 lbs. to the yard. In the upper surface of the rail is a longitudinal groove, 1¼ inch broad and ⅞ inch deep, or thereabouts, to receive the flanges of carriage wheels. The part of the top surface outside the groove is about 1½ inch broad, and is the rolling surface. The part inside the groove is corrugated with shallow transverse grooves, in order to enable carriage wheels to be easily pulled obliquely across them when required. From the under surface of the rail there projects downwards a longitudinal rib, with a flat flange at bottom. This rests upon a properly prepared bed of concrete or other firm material. The spaces between and alongside the rails are filled with granite pavement, or other suitable covering.

(See Appendix, pp. 791, 799, and 809; also Clark, *On Tramways.*)

SECTION IV.—*Of Railways.*

428. **Resistance of Vehicles on a Level.**—Let f be the proportion of the resistance on a level to the gross load, expressed as a fraction; then resistance in lbs. per ton = 2,240 f. ....(1.)

It is true that the part of the resistance which is due to the displacement and friction of the air must depend, not on the load, but on the dimensions and figures of the vehicles; but our experimental knowledge of the laws of the resistance of the air to bodies so large as railway carriages is scarcely sufficient to enable us to calculate that resistance separately with such precision as to make the result of the computation practically useful.*

The co-efficient of resistance on a level, f, consists of two parts; one representing the effect of friction, which is independent of the speed; the other representing the effect of concussion and of the resistance of the air, which increases with the speed. The law according to which the latter part of the co-efficient of resistance increases is still uncertain, owing to the irregularities of the results of experiment. According to one formula (founded on experiments by Mr. Gooch), it is insensible up to a speed of about 10 miles an hour, and then increases nearly in the simple ratio of the excess of the speed above that limit. According to another formula (that of Mr. D. K. Clark), it is nearly proportional to the square of the speed; and both those formulæ agree, in a rough way, with experiment.

The following are the formulæ in question, in each of which V denotes the velocity in miles an hour:—

$$\text{Co-efficient of resistance, } f = \cdot 00268 \left(1 + \frac{V - 10}{20}\right); \quad (2.)$$

$$\text{Resistance in lbs. per ton, } 2{,}240 f = 6 \left(1 + \frac{V - 10}{20}\right); \quad (2\text{ A.})$$

$$\text{Co-efficient of resistance, } f = \cdot 00268 \left(1 + \frac{V^2}{1{,}440}\right); \ldots (3.)$$

* The following are two alternative formulæ by the late Mr. Wyndham Harding and Mr. Scott Russell, in which separate expressions are given for the resistance of the air. In the first formula that resistance is assumed to be proportional to the area of frontage of the train; in the second, to its volume.

T denotes the weight of the train, *in tons*.
V, its velocity, in miles an hour.
A, its area of frontage, in square feet.
B, its volume, in cubic feet; then

$$\text{resistance in lbs.} = \left(6 + \frac{V}{3}\right) T + \frac{V^2 A}{400}; \text{ or}$$

$$= \left(6 + \frac{V}{15}\right) T + \frac{V^2 B}{50{,}000}.$$

$$\text{Resistance in lbs. per ton, } 2{,}240\, f = 6 + \frac{V^2}{240}. \quad (3\text{ A.})$$

Carriages have been made and used in which the co-efficient of resistance was as small as ·002, or about $4\frac{1}{2}$ lbs. per ton, at velocities not exceeding 12 miles an hour, the resistance being sensibly constant at such velocities.*

The formulæ 2, 2 A, 3, 3 A, are applicable to good railway carriages with springs, in trains drawn by an engine at an uniform speed on a well-made line, in good repair, with easy curves, and in moderately calm weather; the experiments on which they are founded having been made under those circumstances, and the resistance determined by means of a dynamometer between the engine and the train.

Another mode of determining the resistance of a carriage on a railway is to start it off at a considerable speed, and allow it to come gradually to rest by its own resistance; but in this mode of experimenting, although the friction is the same as in the other mode, the resistance arising from concussion is considerably less, because much of the vibration originates with the engine.†

The absence of springs augments that part of the resistance which increases with the velocity; but wagons without springs are used only at very low speeds.

The following are some examples of resistances per ton at different speeds, calculated by the two formulæ respectively:—

Speed in miles an hour, V =	10	15	20	30	40	50	60
f by equation 2,...............	·00268	·00335	·00402	·00536	·00670	·00804	·00938
2,240 f by equation 2 A,....	6	7½	9	12	15	18	21
f by equation 3,...	·00287	·00310	·00342	·00435	·00565	·00733	·00938
2,240 f by equation 3 A,. ..	6·4	6·9	7·7	9·7	12·7	16·4	21

Mr. D. K. Clark considers that his experiments indicate that the resistance on a level, given by equations 3, 3 A, is liable to be

* See Rankine *On Cylindrical Wheels on Railways;* also Wood *On Railroads.*

† To ascertain the resistance of a vehicle by experiments on its gradual retardation stones or other marks are to be dropped from the carriage at equal intervals of time (say of t seconds each), and the distances between those successive marks measured. Let x_1, x_2, x_3, x_4, &c., be those distances in feet. i, the sine of the inclination. Then the velocities at the end of

t, $2\,t$, $3\,t$, &c., seconds, are nearly,

$$v_1 = \frac{x_1 + x_2}{2\,t},\ v_2 = \frac{x_2 + x_3}{2\,t},\ v_3 = \frac{x_3 + x_4}{2\,t}, \text{ \&c., in feet per second.}$$

Let v_n and v_{n+1} denote the velocities at the end of $n\,t$ and $(n+1)\,t$ seconds respectively. Then the co-efficient of resistance at the end of $n\,t$ seconds is nearly,

$$f = \left\{(v_n - v_{n+1}) \div 32{\cdot}2\, t\right\} \mp i \text{ according as the gradient is } \begin{cases}\text{ascending} \\ \text{descending.}\end{cases}$$

exceeded in the following proportions, from various occasional causes:—

From a road ill laid, or in bad repair, ...	40	per cent.
From resistance on curves, ...	20	"
From strong side winds, ...	20	"
Total, ...	80	"

It may be held, however, that all those causes of increased resistance are seldom combined at one time and place; and that 50 per cent. is a liberal allowance for *contingent resistances.*

The friction of good ordinary mineral wagons, at low speeds, may be estimated as ranging from

8 lbs. per ton, or ·00353
to 10 lbs. per ton, or ·00446

and as being on an average about

9 lbs. per ton, or ·00402, or 1-250th nearly. (See p. 801.)

429. **Proportion of Gross to Net Load.**—In the following statement the ordinary proportions of the weight of goods and mineral wagons to the loads which they carry are given on the authority of Mr. D. K. Clark; and from those proportions are deduced the proportions of gross to net load in goods and mineral trains:—

	Wagon ÷ Net Load.	Gross Load ÷ Net Load.
Well made open wagons, ...	$\frac{1}{2}$	$1\frac{1}{2}$
Well made covered wagons, ...	$\frac{3}{4}$	$1\frac{3}{4}$
Clumsy wagons, ...	1	2

In computing the gross load to be drawn behind a locomotive engine which has a tender, the weight of the tender (see p. 791) is to be added to that of the wagons and their load.

Passengers without luggage may be estimated at about 15 or 16 to the ton, and with luggage, about 10 to the ton (but this last is an uncertain estimate). In a passenger train the gross load may be roughly estimated at about three times the net load. Many of the fast trains are now composed of long and heavy vehicles; for some data in regard to this, see pp. 794 and 798. In light carriages on horse-worked railways the gross load need not exceed double the net load.

430. The **Tractive Force** which the prime movers on railways exert will here be considered so far as it is connected with the question of gradients. The prime movers commonly employed on

railways are, gravity, horses, fixed steam engines, and locomotive steam engines. The strength of men and the force of the wind have also been employed, but in isolated experiments only.

I. *Gravity* either assists or opposes the other kinds of motive power on all inclined parts of a railway. It may act as the sole motive power on a descending gradient that is sufficiently steep.

The only case in which gravity acts as a *tractive force* on a railway is that of a "*self-acting inclined plane*," on which a train of loaded wagons descending draws up a train of empty wagons. Let i be the sine of the inclination of the plane, f the co-efficient of resistance of the wagons, T the weight of a train of empty wagons, W the net load of a train. Then the available tractive force at an uniform speed is

$$(i - f)\,(\mathrm{T} + \mathrm{W}).$$

The wire ropes used (see pp. 798 to 800) are usually endless, and lie on a series of sheaves or pullies 7 yards apart. The endless, and lies on a series of sheaves or pullies 7 yards apart. The weight of each sheave is between 20 and 30 lbs.; the weight of the rope (allowing 6 as the factor of safety) *per foot of its length* should be 1-4500th of the greatest working tension. Let R be the weight of the rope and pullies; their total resistance is usually estimated at about 1-20th of their weight, and the resistance of the train of empty wagons is $(i + f)$ T. In order that the tractive force may simply balance the resistances, we must have

$$(i - f)\,(\mathrm{T} + \mathrm{W}) = \frac{\mathrm{R}}{20} + (i + f)\,\mathrm{T}; \ldots\ldots\ldots\ldots (1.)$$

and the inclined plane will not work unless the inclination is steeper than that given by solving the above equation; that is to say,

$$i \text{ must be greater than } \left\{ \frac{\mathrm{R}}{20} + f\,(\mathrm{W} + 2\,\mathrm{T}) \right\} \div \mathrm{W} \ldots . (2.)$$

Assume $f = \cdot 004$, $\mathrm{T} = \mathrm{W} \div 2$; then

$$i \text{ must be greater than } \left\{ \frac{\mathrm{R}}{20\,\mathrm{W}} + \cdot 008 \right\} \ldots\ldots (2\text{ A.})$$

The excess of steepness above this limit causes an excess of tractive force above resistance, which produces accelerated motion. The acceleration may be allowed to go on so long as the velocity does not exceed a safe limit; so soon as that limit has been attained, the surplus tractive force must be counteracted by the use of the

brake.* The wear of wire ropes is from 67 to 100 per cent. per annum.

II. *Horses.*—An animal produces its greatest day's work when working for eight hours per day, and with a certain definite speed and tractive force.

Let P_1 denote the tractive force corresponding to the greatest day's work;
- P, any other tractive force;
- V_1, the speed corresponding to the greatest day's work;
- V, any other speed;
- T, the time, in hours per day, during which the exertion is kept up.

Then the following formula is approximately true, for efforts and speeds not greatly differing from P_1 and V_1, and for times not greatly exceeding eight hours per day:—

$$\frac{P}{P_1} + \frac{V}{V_1} + \frac{T}{8} = 3. \qquad (3.)$$

For a good average draught horse the following data are nearly correct:—

$P_1 = 120$ lbs.
$V_1 = 3{\cdot}6$ feet per second, or about $2\frac{1}{2}$ miles an hour.

For a high-bred horse of average strength and activity it is difficult to assign the values of P_1 and V_1, for want of sufficient data. The following values agree in a general way with some of the results of experience in the traction of stage coaches and of light railway carriages:—

* It is seldom necessary to enter into detailed calculations as to the effect of acceleration on a self-acting inclined plane. It may sometimes, however, be desirable to do so, where the declivity is so slight that there is a doubt whether the velocity attained will be sufficiently great to enable a pair of trains to traverse the inclined plane without inconvenient delay.

To find the time occupied in traversing the plane unimpeded, let M denote the total weight of the rope and sheaves, and of both trains, together with one-half of the weight of the pullies. Let F denote the *excess* of the tractive force above the resistance. Let L be the length of the plane; then

$$\text{time in seconds} = \sqrt{\frac{L\,M}{16\,F}} \text{ nearly.}$$

The mean velocity may of course be found by dividing L by this time; and the greatest velocity acquired is double of the mean velocity.

$\therefore P_1 = 64$ lbs.
$V_1 = 7 \cdot 2$ feet per second, or about 5 miles an hour.

The following are examples:—

T, hours per day,.........	4	4	4	1	1	1	1
V, miles an hour, ...	5	$7\frac{1}{2}$	10	5	$7\frac{1}{2}$	10	$12\frac{1}{2}$
P, tractive force, $= 64\left(3 - \frac{T}{8} - \frac{V}{5}\right)$, or	96	64	32	120	88	56	24

It may be observed that the preceding data and calculations have reference to *average* speeds, and that the horse may occasionally be required to exert from once and a-half to double the efforts above stated, provided that he is allowed to slacken his speed during the increased effort, and that the additional exertion is kept up for a short time only.

III. *Fixed Steam Engines* are employed for the most part on short distances, where the speed is moderate and the inclination steep. Their power is usually applied to an endless rope running on sheaves, like that of a self-acting inclined plane (p. 636). The steam engine is placed at the top of the ascent, and drives a large horizontal cast iron pully, from 5 to 10 feet in diameter, having three or four grooves in its rim. This is called the *driving pully*. At a short distance in front of that pully (that is, in the down-hill direction) is a pully one or two feet smaller in diameter, and with one groove fewer in its rim. This is called the *straining pully:* it rests on a small four-wheeled truck, and is pulled away from the driving pully by a chain and weight, the weight being sufficient to give the requisite tension to the rope, which is carried round the grooves of the two pullies. At the foot of the inclined plane the rope passes round a third horizontal pully, as large as the driving pully.

The engine works to the best advantage, and the rope is least strained, when one train is ascending and another descending at the same time.

The greatest tension on the rope is found as follows:—

Let P denote the greatest tractive force required to overcome gravity, and the friction of the train, rope, and sheaves, calculated as in p. 636. About *one-third* of this will be the tension required at the descending side of the rope, to give sufficient "*bite*" or adhesion between it and the driving pully, so that the greatest working tension at the ascending side of the rope will be about.

$$1 \cdot 33 \text{ P};* \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.)$$

and its weight per foot, if it is made of strong charcoal iron wire, should be 1-4500th of this. The pull upon the axis of the straining pully should be about

$$2 \cdot 74 \text{ P}.* \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.)$$

To find the indicated horse-power of the engine, let v be the velocity of the rope in feet per second (= velocity in miles an hour × 1·466); then

$$\text{I} \cdot \text{H} \cdot \text{P} \cdot = \frac{1 \cdot 25 \text{ P } v}{550} = \frac{\text{P } v}{440}; \quad \ldots\ldots\ldots\ldots\ldots (6.)$$

the multiplier 1·25 being introduced on the supposition that the friction of the steam engine wastes one-fifth of the indicated power. (As to "Wire Tramways," see pp. 791 and 799.)

Another mode of transmitting the power of the fixed engine to the train is to employ the engine, by means of a large fan, to exhaust air from or blow air into a tube, along which a piston is propelled towards or from the engine-station by the excess of the pressure behind it above the pressure in front of it. The tube is a brick tunnel, with rails laid along the bottom, on which the wheels of the carriage run; and the piston is a shield fixed on the end of the carriage next the blowing engine, and having enough of clearance round its edge to prevent rubbing against the brickwork. The edge of the shield has a cloth fringe to diminish leakage. For conveying parcels the tunnel is about 3 feet in diameter; and the apparatus is called the "Pneumatic Dispatch."

IV. *Locomotive Engines.*—The tractive force of a locomotive engine is in general limited, not by the power which the engine is capable of exerting—for that is almost always more than sufficient to draw any load that it ever has to convey—but by the "*adhesion*," as it is called, or force which prevents the driving wheels from slipping on the rails.

The adhesion is equal to the weight which rests on the driving wheels, multiplied by a co-efficient which depends on the condition of the surface of the rails; being greatest when they are clean and dry, and least when they are wet and greasy, or covered with ice.

* These calculations are made on the supposition that the co-efficient of friction between the wire rope and the driving pully is ·15, that there are three grooves in the driving pully, and that the tension is made just sufficient to prevent slipping. In practice, however, it is not uncommon to strain the rope till the tension at the descending side is equal to the tractive force; and in that case

greatest tension at the ascending side = 2 P;
pull on the axis of the straining pully = 5·7 P.

On an average, the adhesion of a locomotive engine may be estimated at about one-seventh of the load on the driving wheels; for by sprinkling sand on the rails when they are slimy, or if they are icy, directing jets of steam on them, it may in general be prevented from falling below that amount.

Locomotive engines are seldom made with fewer than six wheels. Those which are intended for the propulsion of comparatively light trains at high speeds may have one pair of driving wheels of from 5 feet 6 inches to 7 feet 6 inches, and sometimes even 8 feet in diameter. The best position for the shaft of that pair of wheels is nearly under the centre of gravity of the engine, in which case, by proper adjustment of the springs, it can be made to bear any proportion of the weight from $\frac{1}{3}$ to $\frac{1}{2}$.

Locomotive engines, although in their essential characteristics having varied little since their introduction, still (owing to the increasing demands for speed and great haulage power and the various features of railway tracks) differ very much in detail from early types.

In the earlier days the tender was always a necessary attachment, and still is when long runs are to be made; but now, in many cases, tank-engines are employed for short runs, where the water can be carried alongside of the boiler as well as coal behind the footplate. Passenger engines are now of such weights that from 13 to 16 tons of the weight come upon each pair of driving wheels. In express passenger engines the driving wheels may be 7 feet diameter, whilst in local service trains and in goods engines 4 or 5 feet diameter is used.

Steam pressures of 150, 175, and 200 lbs. per square inch are used, and in some cases the compound system of engine has been adopted where long runs without much change of gradient obtains.

Some of the express passenger locomotives will take a train, weighing fully 200 tons including the weight of the engine, over long distances at the rate of about 50 miles per hour, on the average, the coal burned per mile being about 45 lbs. Such engines may be considered capable of working up to about 1,000 indicated horse-power. The tractive resistance at this speed becomes about 20 lbs. per ton of weight drawn. (See also pp. 634 and 802.)

WEIGHTS OF ENGINES WITH SEPARATE TENDERS.

(*The Tender weighs with water about 30 tons.*)	*Tons.*
Passenger locomotives,	40 to 55
Goods locomotives, from four to six wheels coupled,	60 to 65

Weights of Tank Engines, carrying Fuel and Water.

	Tons
For light traffic on branch lines..................	20 to 30
For heavy traffic on steep inclined planes......	60 to 80

In comparing the tractive force of a locomotive engine, as limited by adhesion, with the resistance and gravity of the train which it is to draw, it is obvious that the resistance due to friction and concussion *of the engine itself* is to be left out of account; for that resistance does not constitute a backward pull on the engine, tending to make the driving wheels slip.

It appears, then, that the *available tractive force* of a locomotive engine in ascending a given inclined plane, which must be at least equal to the resistance of the heaviest train that it has to draw, is to be found by subtracting from the adhesion that component of the weight of the engine which acts as a resistance to its ascent; that is to say,

Let E denote the total weight of the engine;

q E, that part of the weight which rests on the driving wheels;

i, the sine of the inclination of the railway;

P, the available tractive force; then

$$P = \left(\frac{q}{7} - i\right) E \text{ ....(7.)}$$

The following are examples:—

		q.	$\frac{q}{7} - i$.
Passenger engines, one pair of driving wheels,................	from	·33	·048 — i
	to	·5	·071 — i
Goods engines, two pairs of wheels coupled,.........................	from	·67	·095 — i
	to	·75	·107 — i
Goods engines, all wheels coupled,............		1	·143 — i

By an invention of Mr. Ramsbottom's, the tender of a locomotive is made to supply itself with water while in motion, through a tubular scoop which dips into a long water-trough lying between the rails. The speed should be at least 22 miles an hour, to enable the apparatus to work. (See pp 802, 809, and 810.)

431. **Ruling Gradients of Railways.**—The general nature of a ruling gradient, and of the principles according to which it is determined, have been explained in Article 415, p. 622.

Self-acting inclined planes and *fixed engine inclined planes* are exceptional cases, which are not comprehended under the general principles according to which ruling gradients are determined.

Horse-power is applicable to lines of short length and light traffic

only. The tractive force which a horse can exert under various circumstances has been stated in Article 430, Division II., p. 637. The mean resistance of the goods and mineral wagons on a level may be taken at 1-250th of the gross load, or 9 lbs. per ton; and if the passenger carriages are carefully constructed, in a manner specially suited to the traffic, their resistance may be taken at 1-500th, or $4\frac{1}{2}$ lbs. per ton, on a level straight line, and ·00268, or 1-373d, or 6 lbs. per ton, on a level line with a moderate proportion of curves in its course. It appears from experience that gradients of from 1 in 100 to 1 in 70 may be surmounted without auxiliary power, provided they do not extend to a distance of more than two miles, or thereabouts, at a stretch, and that the horse is not urged to a higher speed than he naturally assumes; but for longer ascents it is advisable, if possible, to limit the steepness to 1 in 200. On ascents of from 1 in 50 to 1 in 10, or steeper, either the load drawn by one horse on other parts of the line should be divided between two, or an auxiliary horse should be harnessed to each carriage or train, and the speed should not exceed a walking pace. Steep ascents for very short distances may sometimes be surmounted by taking a "race" at them.

In fixing the ruling gradient of a *locomotive* railway, it is not to be supposed that rules deduced from the general principles already explained are to be held as absolutely binding. Their proper use is to guide the engineer, when no cause exists sufficient in his judgment to warrant a deviation from them.

This being understood, it appears that there are four things to be adapted to each other,—the greatest load of a train, the least speed of conveyance in ascending declivities, the description of engine, and the ruling gradient; that is, the steepest gradient up which the ordinary traffic is conveyed by the ordinary engines of the line, without the aid of auxiliary engines specially adapted to steep inclined planes. The adaptation of those four things to each other is indicated by the following equation, in which Mr. Clark's formula, Article 428, equation 3, p. 633, is adopted for the resistance of the train:—

Let E denote the weight of the engine (see Article 430, p. 640).

q E, the part of that weight which rests on the driving wheels (see Article 430, p. 641).

T, the gross weight of the train and tender (if there is a tender). As to the proportion of gross to net load, see Article 429, p. 635; as to the weight of the tender, see Article 430, p. 640.

V, the least speed in miles per hour at which the conditions of the traffic will admit of the ruling gradient being ascended.

i, the sine of the ruling gradient (whose inclination in ordinary terms will be described as 1 in $\frac{1}{i}$); then

$$\left(\frac{q}{7}-i\right) E=\left\{\cdot 00268\left(1+\frac{V^2}{1,440}\right)+i\right\} T \ldots\ldots(1.)$$

From this equation are deduced the following formulæ: given q, i, V, to find the ratio of the weight of the engine to that of the train and tender; also the reciprocal of that ratio—

$$E \div T=\frac{\cdot 00268\left(1+\frac{V^2}{1,440}\right)+i}{\frac{q}{7}-i}; \quad (2.)$$

$$T \div E=\left(\frac{q}{7}-i\right) \div\left\{\cdot 00268\left(1+\frac{V^2}{1,440}\right)+i\right\}; \quad (3.)$$

given E, q, T, i, to find the speed of ascent V;

$$V=733 \sqrt{\left\{\left(\frac{q}{7}-i\right) \frac{E}{T}-\cdot 00268-i\right\}}; \ldots\ldots(4.)$$

given E, q, T, V, to find the ruling gradient i;

$$i=\left\{\frac{q E}{7}-\cdot 00268\left(1+\frac{V^2}{1,440}\right) T\right\} \div(E+T). \quad (5.)$$

According to Mr. Clark's allowance of 50 per cent. for occasional or contingent resistances referred to in Article 428, p. 635, ·00402 may occasionally have to be substituted for ·00268 in the preceding formulæ.

The following may be taken as examples of the results of such computations, the formula employed being equation 3:—

EXAMPLE.		I.	II.	III.	
Speed in miles an hour,		24	18	12	
		Tons.	Tons.	Tons.	
Weight of engine,		20	30	30	
Number of driving or coupled wheels,		2	4	all	
		Tons.	Tons.	Tons.	
Load on driving or coupled wheels,		10	21	38	
Weight of tender,		10	12	15	
Ascending gradient.	in 50,	33	91	145	Gross load of train
	in 80,	63	154	238	
	in 100,	79	191	293	
	in 133·3,	104	245	373	
	in 200,	142	332	595	

The thing to be principally considered in fixing a ruling gradient is the traffic. This having been ascertained, so as to determine the probable gross load of the several descriptions of passenger and goods trains, and the speed at which they are to run up the steepest parts of the line, the ruling gradient is to be fixed so that engines with not more than about 5 tons of load on each wheel may be able to draw the trains.

It is in general bad economy to incur heavy works in order to ease the ruling gradient, merely for the sake of enabling light engines to convey a heavy traffic; but where the traffic is light, and moderate gradients can be obtained without heavy works, light engines may be used with advantage.

431 A. The **Action of Brakes** may have to be considered in connection with questions respecting gradients.

The immediate effect of applying brakes is to stop wholly or partially either some or all of the wheels of the train, so that they slide instead of rolling on the rails; and the increased resistance thus produced stops the movement of the train in the course of a *time* proportional directly to the speed and inversely to the resistance, and of a *distance* proportional directly to the square of the speed and inversely to the resistance. The *distance* in the course of which the train is stopped is of more importance practically than the time, and is found as follows:—

Let f'' be the proportion which the resistance produced by the brakes bears to the weight of the train;

v, the speed in feet per second; then

$$\text{distance in feet on a level} = v^2 \div 64 \cdot 4 f'' . \ldots\ldots\ldots (1.)$$

For practical purposes it is more convenient to state the velocity in miles an hour. Let V denote that velocity; then

$$\text{distance in feet on a level} = V^2 \div 30 f'' \text{ nearly.} \ldots . (2.)$$

There are self-acting brakes, operated upon by the buffers, by mechanism worked by steam, or otherwise, which act on all the wheels at once.* For such brakes it may be considered that

$$f'' = \cdot 14, \text{ nearly.} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.)$$

It is not considered desirable to stop a train much more suddenly than these brakes do, lest an injurious shock should be produced.

For ordinary brakes, worked by hand in carriages called "brake vans," the value of f'' may be estimated as ranging

$$\text{from about } \cdot 031 \text{ to } \cdot 023; \ldots\ldots\ldots\ldots\ldots\ldots (4.)$$

* See Appendix, pp. 789 and 796.

so that they stop the trains in distances ranging from 4½ to 6 times the distances required by brakes that act on all the wheels.*

The following are some of the results of those data, calculated in round numbers, as precision of calculation is useless in this case:—

Speed in miles an hour, ...		10	20	30	40	50	60
Distance in feet required for stopping the train on a level, with brakes acting on all the wheels, ...		24	96	216	384	600	864
With ordinary brakes,...	from	108	432	972	1728	2700	3888
	to	144	576	1296	2304	3600	5184

On a gradient ascending at the rate of 1 in $1 \div i$, the resistance available for stopping the train becomes $f'' + i$, and it is stopped in so much the shorter distance.

On a gradient *descending* at the rate of 1 in $1 \div i$, the resistance available for stopping the train is diminished to $f'' - i$, and the distance required for stopping it becomes,

$$\text{distance on a level} \times f'' \div (f'' - i). \text{ ....(5.)}$$

432. **Gradients with Auxiliary Power.**—Where an inclined plane occurs steeper than the ruling gradient, auxiliary power may be applied either by attaching an additional locomotive to each train, or by having special locomotives of great weight and power to draw the trains up, or by using a fixed engine and rope. The economy of the use of auxiliary power depends mainly on the constancy with which it can be kept at work; and this depends on the nature of the traffic.

The locomotive engines used for this purpose are usually tank engines, in order that the weight producing adhesion may be as great as possible.

433. **Power Exerted by Locomotive Engines.**—Besides drawing the train, the locomotive engine has to overcome the resistance of its own wheels and axles, and of its own mechanism. If the power or mechanical energy expended in overcoming this additional resistance, while the engine travels over a given distance, be divided by that distance, there is obtained the additional train-resistance which would be equivalent to the resistance of the engine; and this being added to the resistance of the tender and train, gives the *gross resistance* of the engine, tender, and train. Various rules have been proposed and tried for computing the additional resistance of the engine.

The following rule is founded on the principle that the resistance of the engine consists of two parts; the first, being the resistance of

* See Addendum, p. 796.

the engine *as a carriage*, is the same with that of a train of the same weight; the second, being the resistance caused by the strain on the mechanism, bears a certain proportion to the whole resistance of the engine and train, whether arising from friction, concussion, or gravity; and that proportion appears to be about one-third. That principle is expressed by the following formula for the gross resistance R, of an engine whose weight is E, drawing a tender and train whose gross weight is T, at the speed of V miles an hour up an incline of 1 in 1 $\div$ i:—

$$R = \frac{4}{3}(T + E)\left\{ \cdot 00268 \left(1 + \frac{V^2}{1{,}440}\right) + i \right\}; \ldots\ldots (1.)$$

or if R is in lbs., and T and E in tons,

$$R = (T + E) \cdot \left\{ 8 + \frac{V^2}{180} + 2{,}987\, i \right\}.^* \ldots\ldots (2.)$$

Under unfavourable circumstances ·00402 may occasionally have to be substituted for ·00268, and $12 + \frac{V^2}{120}$ for $8 + \frac{V^2}{180}$.

For a descending gradient, each term in i is to be subtracted instead of added.

The energy exerted by the engine per minute, in foot-pounds, is the product of the effort or gross resistance in pounds and speed in feet per minute; that is to say,

$$88\ V\ R; \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.)$$

(one mile an hour being 88 feet per minute). The *indicated horse-power* is

$$\frac{88\ V\ R}{33{,}000} = \frac{V\ R}{375} \ldots\ldots\ldots\ldots\ldots\ldots (4.)$$

Let A be the area of each of the two pistons of the engine, in square inches; p, the *mean effective pressure*, in lbs. on the square inch; c, the circumference of the driving wheels, in feet; l, the length of stroke of the pistons, also in feet; also, let d be the diameter of the pistons, and D that of the driving wheels; then

* The above formula differs from that of Mr. D. K. Clark in the following respects:—One-third is added to the whole resistance of the engine and train, considered as carriages, whether arising from friction, concussion, or gravity; whereas, in Mr. Clark's formula, one-third is added to the friction, two-fifths to the resistance from concussion, and nothing to the resistance from gravity.

$$
\left.
\begin{array}{l}
p\,\mathrm{A} = \dfrac{c\,\mathrm{R}}{2\,l};\ \text{and} \\[2ex]
p = \dfrac{c\,\mathrm{R}}{4\,l\,\mathrm{A}} = \dfrac{\mathrm{D\,R}}{l\,d^2}.
\end{array}
\right\} \ldots(5.)
$$

The *mean speed* of the pistons is $176\,l\,\mathrm{V} \div c$;

The *volume swept through by the piston per minute, in cubic feet.* $\Big\}\ \dfrac{22\,l\,\mathrm{V}\,\mathrm{A}}{9\,c} = \dfrac{11\,l\,\mathrm{V}\,d^2}{18\,\mathrm{D}}$.

EXAMPLE.	I.	II.	III.
Speed in miles an hour,......	24	18	12
Ascending gradient, ...	1 in 133·3	1 in 133·3	1 in 80
Weight of engine,...	20 tons	30 tons	30 tons
Weight of tender,.............	10 ,,	12 ,,	5
Weight of train,...............	104 ,,	245 ,,	238 ,,
Circumference of driving wheel,....................	20 feet	15 feet	14 feet
Stroke of pistons, ...	2 ,,	2 ,,	2 ft. 2 in.
Area of each piston, ...	200 sq. in.	226 sq. in.	253 sq. in.
Effort or gross resistance, ...	4,502 lbs.	9,211 lbs.	13,057 lbs.
Mean effective pressure in lbs. on the square inch,	56·3 ,,	76·7 ,,	83·4 ,,
Mean speed of pistons, feet per minute,.........	422·4	422·4	328·85
Volume swept through by pistons in cubic feet per minute,....................	1173·3	1325·9	
Indicated horse-power,......	288	444	418

The mean effective pressure of steam in the cylinder is regulated by the effort required to overcome the resistance, as shown by the formulæ and calculations just given. The pressure of the steam in the boiler exceeds the mean effective pressure in the cylinder in a proportion depending on the extent to which the steam is worked expansively, and various other circumstances. The pressures used reach nearly to 200 lbs. per square inch, which appears to favour economy. (See pp. 794 and 798.) The compound system has been tried in locomotives both in this country and abroad; the results are most favourable on long runs with little variation of gradient.

The speed at which the engine runs when exerting a given effort is regulated by the quantity of steam at the required pressure which the boiler is capable of producing; which quantity depends on the

quantity of fuel that can be burned in the furnace in a given time, and the efficiency of that fuel in producing steam.

The consumption of fuel by locomotive engines, per indicated horse-power per hour, may be estimated as ranging from 3 to 5 lbs., and the evaporation from 7 to 9 lbs. per lb. of fuel. The whole area of heating surface in ordinary engines varies from 800 to 2,000 square feet; and the area of heating surface for each lb. of fuel burned per hour varies from about half a square foot to 1½ square foot, and is on an average about one square foot. (See also p. 802.)

The action of the blast-pipe gives to the locomotive engine the power of adapting its consumption of fuel to the work which it has to perform, within certain limits. Hence the rapid consumption of fuel by heavy and powerful engines, in ascending steep inclined planes, is to a great extent compensated by the saving which takes place in descending.

For details of the construction of locomotive engines, and of the action of the fuel and steam in them, see Jamieson *On the Steam Engine*, Sauvage, *La Machine Locomotive*, Woods *On Compound Locomotives*, Fletcher *On Steam Locomotion on Common Roads*, D. K. Clark *On Railway Machinery*, Z. Colburn *On Locomotive Engineering*, and the Author *On Prime Movers*.

434. **Curves.**—1. *Additional Resistance on Curves.*—Curves on a line of railway increase the resistance to an extent which is somewhat uncertain. From experiments made by Lieutenant David Rankine and the author on light passenger carriages with truly cylindrical wheels, having a resistance of ·002, or 4½ lbs. per ton, on a level straight line, it appeared that the additional resistance was

$$\left.\begin{array}{lll} \text{in fractions of the load,} & 3{\cdot}3 & \div \text{ radius in feet;} \\ \text{or,} & {\cdot}000625 & \div \text{ radius in miles;} \\ \text{or in lbs. per ton,} & 1{\cdot}4 & \div \text{ radius in miles.*} \end{array}\right\} \quad (1.)$$

So long as the practice prevailed of *tapering* the tires of wheels to a considerable extent, the resistances on curves and straight lines appear to have been nearly equal; the tapered or conical form somewhat diminishing the resistance on curves, while it considerably increased that on straight lines, by causing the carriages to move in a serpentine course, and so augmenting the resistance due to concussion. But since the taper of the wheels has been in some cases done away with, and in others made very slight (about 1 in 40), the resistance on straight lines has become sensibly less than on curves.

Some experiments on American railways by Mr. Latrobe give, for the resistance due to curvature, the following results:—

* Rankine *On Cylindrical Wheels*, 1842.

$$\left.\begin{array}{lll} \text{in fractions of the load,} & 1{\cdot}36 & \div \text{ radius in feet;} \\ \text{or,} & {\cdot}000258 & \div \text{ radius in miles;} \\ \text{or in lbs. per ton,} & 0{\cdot}58 & \div \text{ radius in miles;} \end{array}\right\} \quad (2.)$$

The smallness of these results, compared with those given in the formulæ (1), may perhaps be owing to the use of "bogey" carriages.

II. *Adaptation of Vehicles to Curves.*—Both engines and carriages are adapted to sharply curved lines of railway by means of the "bogey"—a truck capable of turning about a pivot into various positions relatively to the carriage or engine which it supports. A long passenger carriage is supported on two four-wheeled bogeys, one near each end. The "Fairlie" locomotive engine is supported in the same way; each of the bogeys has its wheels driven by an independent engine, and the boiler and furnace are in the middle. (See *The Engineer*, 1870, vol. xxix., p. 389.) Instead of bogeys and pivots, Mr. W. B. Adams uses a pair of axle-boxes for the leading wheels, sliding in curved guides, whose centre is at a point near the middle of the carriage. By the aid of such contrivances engines and carriages are enabled to pass round curves of radii as small as 3½ chains (231 feet). On British railways, curves of sharper radii than 10 chains are of rare occurrence. (See ADDENDA, p. 791.)

III. *Cant of Rails of a Curve.*—This term denotes the transverse slope which is given to the surface of the rails of a curve, in order to counteract the tendency of the carriages to go straight forward, and so to leave the curve. That tendency arises from three causes,—from the centrifugal force, from the parallelism of the axles, and from the slip of the wheels.

Let v be the velocity of a train in feet per second, moving round a curve of the radius r in feet, then its *centrifugal force* bears to its weight the proportion of

$$\frac{v^2}{32{\cdot}2\, r} : 1; \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.)$$

and this is the ratio which the *cant*, or elevation of the outer above the inner rail of the curved line of rails, must bear to the GAUGE, or transverse distance between the rails.

If V be the speed in miles an hour,

$$\text{cant for centrifugal force} = \text{gauge} \times \frac{V^2}{15\, r} \text{ nearly} \ldots (4.)$$

The clear gauge is—

On British railways, ... 4 feet 8½ inches.

(See p. 798 for various gauges.)

On Irish railways, ... 5 feet 3 inches.

One-half of the cant should be given by raising the outer rail above the level of the centre line, the other half by depressing the inner rail. It is impossible to adjust the cant alike for all speeds; but it is best to adapt it nearly to the highest speed of ordinary occurrence on the line.

For example, suppose that speed to be 40 miles an hour; then the values of the cant for centrifugal force, in inches, are as follows for different gauges:—

Gauge.	Cant for Centrifugal Force, in Inches.
4 feet $8\frac{1}{2}$ inches.	6,000 ÷ radius in feet.
5 " 3 "	6,720 ÷ radius in feet.
6 " 0 "	7,680 ÷ radius in feet.
7 " 0 "	8,960 ÷ radius in feet.

The tendency to leave the line which arises from the axles being parallel, instead of radiating from the centre of the curve, cannot easily be distinguished from that due to the next cause.

The tendency to leave the line which arises from the slip of the wheels is produced in the following way:—The outer rail of any given arc of the curve is longer than the inner rail in the ratio of

$$\text{radius} + \text{gauge} : \text{radius};$$

and while the inner wheel rolls over a given arc of the inner rail, the outer wheel, if it be of the same diameter with the inner wheel, has to slip over a distance equal to the difference of the lengths of the rails. Thus is produced an additional resistance to the advance of the outer wheel of each pair of wheels, tending to make the front end of the carriage swerve outwards. The taper or coning of the wheels was devised to prevent this tendency, by causing the outer wheel to run on a portion of its tire of larger diameter than that on which the inner wheel runs at the same time. It has the disadvantage already mentioned, however, of increasing the oscillation or sideward swinging of trains on straight lines. The tendency to swerve may be corrected in cylindrical wheels by means of an additional cant, which throws the larger proportion of the weight on the inner rail. The additional cant required for that purpose was determined experimentally by Lieutenant David Rankine and the author for carriages moving on a narrow gauge line at speeds of from 3 to 12 miles an hour, and found to be independent of the velocity, and inversely proportional to the radius of the curve; being given by the following formula:—

$$\text{additional cant in feet} = 600 \div \text{radius in feet}; \ldots (5.)$$

$$\text{" " in inches} = 7{,}200 \div \text{radius in feet}; (5\text{ A.})$$

but such additional cant is probably rendered unnecessary by the use of bogeys, or of axle-boxes sliding in curved guides.

IV. *Method of Easing Changes of Curvature.*—Every change of curvature should be accompanied by a change in the cant of the rails; and as changes of the cant of the rails can only be made by degrees, changes of curvature should be gradual also, whether they occur at the junctions of curves with straight lines, or of curves of reverse curvature, or of different radii, with each other.

Two methods of setting out curves with gradual changes of curvature have been practised—one, invented by Mr. Gravatt about 1828 or 1829, consists in the use of the *curve of sines* instead of the circle for railway curves; the other invented by the late Dr. William Froude, consists in adhering to the use of the circle throughout the greater part of the extent of each curve, but introducing at each end of each curve a small portion of a curve approximating to the *elastic curve*, for the purpose of making the change of curvature by degrees. The proper place for referring to these methods would have been Article 63, p. 101, which relates to the ranging and setting out of curves; but although they have been so long in use, no account of either of them was published until after that part of this book was in type. For details respecting them, see *Transactions of the Institution of Engineers in Scotland*, vol. iv., 1860-61.

The curve of sines is that which gives the most gradual variation of curvature; but its use involves the abandonment of circular arcs, which are the most easily and rapidly set out of all curved lines. As Dr. Froude's "*curve of adjustment*" (as it may be called) is easily combined with circular arcs, the most convenient mode of applying it to practice will now be described.

Suppose that a portion of a line of railway, consisting of a curve, or of a series of curves, but beginning and ending with straight lines, is to be set out in such a manner that all changes of curvature shall be gradual.

(1.) Begin by ranging the centre line as a series of circular arcs, according to Method I. of Article 63, p. 102, and marking it with poles or temporary stakes.

(2.) Determine the *length* to be adopted for the "*curves of adjustment*" as follows:—Compute the *cant* required for each of the circular arcs, according to the rules of Division III. of this article, and the several *changes of cant;* observing that the change of cant between a straight line and a curve is simply the cant of the curve; that if two adjacent curves are curved in the same direction, the change is the difference of cant; and that if they are curved in reverse directions, the change is the sum of the two cants.

Multiply the *greatest change of cant* by the reciprocal of the

steepest gradient of adjustment; that is, the greatest *difference of inclination* which can conveniently be given to the outer and inner rail in changing the cant. The result will be the length of each of the curves of adjustment.

According to Dr Froude, the gradient of adjustment should not exceed 1 in 300. Then,

$$\text{Length of curve of adjustment} = 300 \times \text{greatest change of cant.} \quad (6.)$$

(3.) Compute, for each circular arc of the series, the *shift* as follows:—

$$\text{Shift} = (\text{length of curve of adjustment})^2 \div 24 \text{ radius.} \quad (7.)$$

Then shift the poles by which a given circular arc is marked inwards (that is, towards the centre of curvature of the arc) through the distance computed by the above formula. For example, in fig. 276, let A B, B C, be a pair of consecutive circular

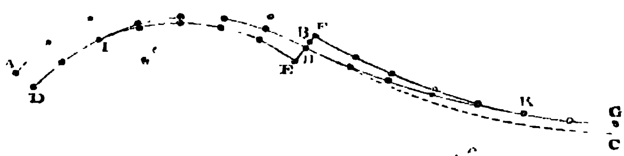


Fig. 276.

arcs, marked by poles, and joining each other at their point of contact B. Let B E, B F, be the *shifts* proper to those two arcs respectively, as computed by equation 7; after all the poles have been shifted, they will mark the arcs D E, F G, having a gap between them at E F, equal to the sum of the two shifts, if the arcs are curved in reverse directions, or the difference of the shifts, if the arcs are curved in the same direction. Straight lines are not to be shifted; so that where a curve joins a straight line, the gap is simply the shift of the curve.

(4.) Set out the "*curve of adjustment*" I H K as follows:—For its middle point, bisect the gap E F in H. For its ends I and K, lay off E I and F K, each equal to half its length, as computed by equation 6. For intermediate points in the division I H, lay off ordinates at right angles from a series of points in the circular arc I E, proportional to the cubes of the distances from I; and for intermediate points in the division K H, lay off ordinates at right angles from a series of points in the circular arc K F, proportional to the cubes of the distances from K.

The following is a formula for calculating those ordinates:—

Let a denote the length I K of the curve of adjustment;
b, the gap E F, or sum of the shifts;
x, the distance, measured on the circular arc, of any point from I or from K, as the case may be;
y, the ordinate; then

$$y = \frac{4\,b\,x^3}{a^3} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(8.)$$

EXAMPLE.—A curve of 20 chains radius (= 1,320 feet), with cant suited to a speed of 40 miles an hour on a narrow gauge line, is to be connected with a straight line.

Cant (see p. 650) = 500 feet ÷ 1,320 = ·3788 foot;
Length of curve of adjustment, a = ·3788 × 300 = 113·6 feet;
Shift for circular arc = $(113·6)^2$ ÷ 24 × 1,320 = ·407 foot.

(As the arc is to join a straight line, this is also the width of the gap b.)

$$\text{Formula for ordinates, } y = \frac{4 \times ·407\,x^3}{(113·6)^3} = ·000,001,11\,x^3.$$

The easing or "humouring" of changes of curvature is performed by rail-layers by the eye, with considerable accuracy, in the case of reverse curves, where a "bit of straight" has been set out to connect the two circular arcs; but no such approximate process is possible at junctions of curves with straight lines, or of curves of unequal radii, curved in the same direction.

V. *Combination of Curves and Gradients.*—As curvature of the line increases the resistance of trains and the danger of jumping off the line at high speeds, it is advisable to avoid very sharp curves on steep gradients, and on parts of the line where the speed is to be very high.

Where sharp curves necessarily occur in the course of a steep ascent, it is advisable, instead of adopting an uniform gradient, to make it slightly steeper on the straight parts of the line, and slightly flatter on the curved parts, in order that the resistance of an ascending train may be as nearly as possible uniform. In the absence of more precise data, formula 1 of Division I. of this Article, p. 648, may be used to compute the resistance due to curvature for engines and carriages without bogeys, and formula 2, p. 649, for those with bogeys.

VI. *Additional Problems in Setting out Curves.*

PROBLEM FIRST.—*To find the radius of a circular arc which shall*

successively touch three straight lines, B D, D E, E C, fig. 277, measure the middle straight line D E, and the acute angles at D and E. Then

$$\text{Radius} = \mathrm{D\,E} \div \left(\tan\frac{1}{2}\mathrm{D} + \tan\frac{1}{2}\mathrm{E}\right); \ldots\ldots\ldots(9.)$$

which having been computed, the curve can be set out by Method I. of Article 63, p. 102.

Problem Second.—*To set out the curve of sines, or harmonic curve,* proposed by Mr. Gravatt. This curve may be used with advantage where there is clear ground and sufficient time to range it. Let B A, C A, be the two straight tangents to be connected

Fig. 277.

Fig. 278

by means of the curve, cutting each other in A. Lay out the straight line A D E, bisecting the angle B A C, and choose in it a point D for the curve to traverse. Lay off the distances.

$$\mathrm{A\,B} = \mathrm{A\,C} = 2{\cdot}75193\ \mathrm{A\,D} \cdot \sec\frac{1}{2}\mathrm{B\,A\,C}; \ldots\ (10.)$$

then B and C will be the ends of the curve. Conceive the chord B E C to represent a semicircle stretched out straight, and divided into 180 degrees, and lay off ordinates at right angles to it proportional to the sines of arcs marked upon it: the ends of those ordinates will be points in the curve. The middle or longest ordinate is,

$$\mathrm{D\,E} = 1{\cdot}75193\ \mathrm{A\,D}; \ldots\ldots\ldots\ldots\ldots\ldots(11.)$$

Let x denote any abscissa B X, measured from one of the ends of the curve; y, the corresponding ordinate X Y; then

$$y = \mathrm{D\,E} \cdot \sin\frac{90^\circ \times x}{\mathrm{B\,E}}; \qquad (12.)$$

the value of the half-chord B E being,

$$B E = 2 \cdot 75193 \; A D \cdot \tan \frac{1}{2} B A C. \qquad (13.)$$

The sharpest curvature occurs at D, where the radius of curvature

$$= D E \cdot \tan^2 \frac{1}{2} B A C. \qquad (14.)$$

If the cant of the rails at D is adapted to this radius, the cant at any other point may be determined with sufficient accuracy for practice by making it vary simply as the ordinate y. The form of the curve is nearly, though not exactly, that of an elastic bow of uniform section, bent by means of a string connecting its ends.

PROBLEM THIRD.—*To connect a circular arc and a straight line, or two circular arcs, which do not touch or cut each other, by means of an elastic curve* (Dr. Froude's curve of adjustment). Fig. 276, p. 652, may be taken to illustrate the case where two arcs curved in reverse directions are to be connected; fig. 279, to illustrate that in which two arcs curved in the same direction are to be connected.

Find the pair of points at which the arcs or lines to be connected are nearest to each other. This is best done by first finding two pairs of points at which the lines to be connected are at equal distances apart; the pair of points required will be midway between those two pairs of points. Let E and F be the pair of points thus found; measure the *gap* E F, then calculate the *half-length of the curve of adjustment* by means of the following formula, in which r and r' denote the radii of the arcs to be connected:—

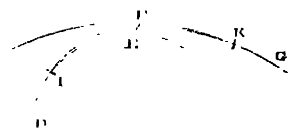


Fig. 279.

$$E I = F K = \sqrt{\left\{ 6 \; E F \div \left(\frac{1}{r} \pm \frac{1}{r'} \right) \right\}}; \qquad (15.)$$

the sign + or — being used in the denominator, according as the directions of curvature are reverse or similar. If one of the lines to be connected is straight, $1 \div r'$ is to be made $= 0$; so that the formula becomes

$$E I = F K = \sqrt{6 \; E F \cdot r}. \qquad (16.)$$

(The ends of the curve of adjustment may also be determined approximately, by finding the two pairs of points at which the distance between the lines to be connected is 4 E F.)

The curve of adjustment is now to be set out by ordinates, as in Division IV. of this Article, p. 652.

VII. *Enlargement of Gauge on Curves.*—In order to enable

trains to pass more easily round curves, it is the practice of some engineers to make the gauge about half an inch wider on them than on straight lines; so that supposing, for example, that the wheels have half an inch of "*play*," or "*clearance*," on straight lines, they will have an inch on curves.

VIII. *Legal Limitations to the Sharpness of Curves.*—According to the Railway Clauses Consolidation Act of 1845, the power of diminishing the radii of curves below the length marked on the parliamentary plan of a railway is thus limited:—if the radius of any curve, as shown on the parliamentary plan, exceeds half a mile (2,640 feet), it may be shortened to any extent which does not reduce it below half a mile; but no radius of such a curve is to be shortened to less than half a mile, nor is any radius shown as less than half a mile to be shortened to any extent. See Trautwine, *Field Practice of Laying Out Circular Curves for Railways*, and M'Kay, *Light Railways.*

435. **Laying out and Formation of Railways in General.**—Subject to the principles respecting gradients and curves which have been explained in the preceding articles of this section, the general principles of the selection of the course and the formation of a line of railway are those which have already been stated in Articles 413, 414, pp. 619 to 622, and the other parts of this work referred to in those articles.

The following principles are specially applicable to railways:—

I. The *Breadth of Formation or Base* depends upon the gauge, or clear distance between the rails of a track, the number of tracks, the clear space between them, the clear space left outside of them for projections of carriages and for men on foot, and the additional space required for the slopes of the "*ballast*," the side drains, &c. The following are examples (see also pp. 792, 798):—

SINGLE LINE.		Narrow Gauge. Ft.	In.
Clear space outside of rail,		4	0
Head of rail		0	2½
Gauge,		4	8½
Head of rail,		0	2½
Clear space outside of rail.		4	0
Least breadth of top of ballast; and least width admissible for archways, &c., traversed by the railway,		13	1½
Spaces for slopes of ballast, and benches beyond them, on embankments,	from	3	10¼
	to	8	10¼
Total breadth of top of embankments,	from	17	0
	to	22	0

For gauges adopted in various countries see p. 798.

In some countries a gauge of about 5 feet is used. (See also p. 798.)

DOUBLE LINE.

		Ft	In.
Clear space outside of rail,		4	0
Head of rail,		0	2½
Gauge,		4	8½
Head of rail,		0	2½
Middle space (called the "six feet"),		6	0
Head of rail,		0	2½
Gauge,		4	8½
Head of rail,		0	2½
Clear space outside of rail,		4	0
Least breadth of top of ballast; and least width admissible for archways, &c., traversed by the railway,		24	3
Spaces for slopes of ballast and trenches beyond them on embankments,	from	3	9
	to	8	9
Total breadth of top of embankments,	from	28	0
	to	33	0

Cuttings are sometimes made of a width at the formation level equal to that of the embankments on the same line; in other cases they have an additional width given to them, amounting sometimes to as much as 9 feet, in order that there may be the more space for the side drains. On the whole, the most common breadths of base for both embankments and cuttings are

for single lines, 18 feet,
for double lines, 30 feet.

Arches over the railway are seldom made of the minimum spans shown by the foregoing tables, except in the case of tunnels. Bridges over railway lines are usually of the following spans:—

over a single line, from 16 to 18 feet;
over a double line, from 28 to 30 feet;

and the same breadths are applicable to cuttings with retaining walls, and rock cuttings with vertical or nearly vertical sides.

II. The *Formation Level* is from $1\frac{1}{2}$ to 2 feet, or thereabouts, below the intended level of the rails, according to the depth of the permanent way (see Article 66, p. 112), and is marked by a line of some distinctive colour on the working section.

III. *Side Slope.*—The formation or base is sometimes made to fall from the centre towards the sides at the rate of about 1 in 60, to facilitate drainage.

IV. *Cross Drains.*—Where the nature of the soil makes cross drains necessary, they may be made by digging small trenches across

the base from 7 to 9 inches deep, and from 3 to 5 yards apart, and filling them with broken stone.

V. *Positions of Stations relatively to Gradients.*—Although it may sometimes be absolutely necessary to have stations in the course of steep gradients, the engineer should, as far as possible, avoid that necessity, because of the difficulty and inconvenience of stopping descending trains, starting ascending trains, and shifting carriages at stations so placed. There is an advantage in having a station at a summit level, because the gradients facilitate the starting and stopping of trains in both directions.

VI. *Legal Limits to Powers of Deviation.*—In Britain, the ordinary limits of deviation as to the situation of a railway are 100 yards in the country and 10 yards in towns to either side of the centre line, as marked on the parliamentary plan, and such limits are marked on the plan. Wider limits, in special cases, are granted by special enactment upon sufficient cause being shown; and the limits may be restricted, at the discretion of the promoters, to any extent consistent with the execution of the work. The ordinary limits of deviation of the level of a railway are 5 feet in the country and 2 feet in towns above and below the level of the upper surface of the rails, as shown on the parliamentary section. Further deviations of level require the sanction of owners of property affected by them, except in the case of embankments and viaducts, which may be lowered to any extent consistent with leaving sufficient headroom for roads.

Gradients less steep than 1 in 100 may be made steeper to an extent not exceeding 10 feet per mile; gradients of 1 in 100, or steeper, may be made steeper to an extent not exceeding 3 feet per mile; gradients may be made flatter to any extent.

As to curves, see Article 431, p. 656. On all these points, see the *Railways' Clauses Consolidation Act*, 1845.

436. **Crossings and Alterations of other Lines of Conveyance.**—I. *General Explanations.*—When the course of a railway crosses that of a previously existing line of land-carriage, the railway may either be carried over or under the existing line by means of a bridge, or across it on the level of its surface. When the line of conveyance to be crossed is a canal or a river, the railway must be carried either over or under it. In order to facilitate such crossings, it may be necessary to alter the level or divert the course of existing lines of conveyance; and in some cases a diversion of the course of an existing line of conveyance may be required independently of any crossing; and for all those purposes, cuttings and embankments are required. The parts of a road whose levels are altered for the purpose of carrying the railway across it, are called the *approaches* of the crossing.

The information which must be given on the section of a proposed railway respecting such alterations of existing lines of communication has already been referred to in Article 14, pp. 14, 15. Proposed diversions of such lines should be shown on the plan, and proposed alterations of width noted. But, as formerly stated, with regard to all matters connected with the preparation of parliamentary plans, reference should be made to the standing orders of parliament themselves, and not to any second-hand account of their provisions.

As to working sections of alterations of existing lines of communication, see Article 66, p. 113.

II. *Legal Limitations affecting Crossings of existing Lines of Conveyance.*—In order fully to understand those limitations, as they are regulated by law, in Britain, the statutes called "Railways' Clauses Consolidation Acts" must be consulted. The following is an outline of the more important of the limitations:—

A. *Level Crossings* of public carriage roads are not lawful unless individually authorized by the special act relating to the particular railway; and in order that they may be so authorized, the engineer must be prepared to show cause for using them instead of bridges, and to prove that they are consistent with the public safety. All level crossings must be provided with gates, which, in ordinary, are to be kept shut across the road. In the case of level crossings of public roads, those gates are to be capable of being closed across the railway when the passage along the road is open; and there must be a lodge or box for a gatekeeper, and a proper system of signals.

B. *Over Bridges* (as bridges for carrying roads over the railway are called) are to have a clear width of roadway between the parapets,

for a turnpike road,... ....of 35 feet,
for any other public carriage road, of 25 feet,

provided the average width of the road between its fences throughout a distance of 50 yards on each side of the centre line of the railway is not less than 35 feet or 25 feet, as the case may be. Should the average width of the road be less than the limit abovementioned, the roadway of the bridge may be made of a width equal to such existing average width, provided that in no case shall the roadway be made of a less clear width than,

for a turnpike road,....... .... 30 feet,
for any other public carriage road, ... 20 „

and that, if at any future period the road should be widened, the railway company shall be obliged to widen the bridge to 35 feet or 25 feet as the case may be.

For *private roads* the prescribed least width is 12 feet; but this may be altered by special agreement between the proprietor of the road and the promoters of the railway.

The *parapets* of all over bridges are to be at least 4 feet high, and the fence of their approaches at least 3 feet high.

C. *Under Bridges* (as bridges for carrying roads under the railway are called) are subject to the same conditions as to width of roadway with over bridges; and those conditions fix the least span of the arch. Its height is subject to the following conditions:—

For a *turnpike road* the clear headroom is to be at least,
at the springing of the arch, 12 feet;
throughout a breadth of 12 feet in the middle of the archway, 16 feet.

For any other *public carriage road* the clear headroom is to be at least,
at the springing of the arch, 12 feet;
throughout a breadth of 10 feet in the middle of the archway, 15 feet.

For a *private road* the clear headroom is to be at least,
throughout a breadth of 9 feet in the middle of the archway, 14 feet.

D. The *inclination* of an altered road is not to be made steeper,
for a turnpike road, than 1 in 30;
for any other public carriage road,... than 1 in 20;
for a private road, than 1 in 16;

provided that the undertakers of the railway shall not be obliged to make the inclination of the altered road easier than its original "mesne" inclination, or than the original "mesne" inclination of the road within a distance of 250 yards from the point where it crosses the centre line of the railway.

(No rule is prescribed for computing the "mesne" inclination of the road; but the following appears to be as little open to objection as any that can be devised. Add together all the rises and all the falls of the portion of the road in question, and divide their sum by its length.)

E. The rules of the general act may be modified or set aside in particular cases by special enactment, upon sufficient cause being shown; and in the case of crossings of private roads, conditions may be settled by agreement between their proprietors and the promoters of the railway.

F. A sufficient *temporary road* must be provided until the permanent road is complete. In many cases it is most convenient to divert the road, and use the original roadway as a temporary road.

G. Works in tidal waters must be sanctioned by the admiralty.

III. The *Least Dimensions of Under Bridges* are virtually fixed by the rules above-mentioned. The following are examples, in which the arches are treated as segmental, that being the best form in the present case. The rise is given as computed by Mr Cotton in his work *On Railway Engineering in Ireland*.

The power to make roadways of less than the prescribed widths of 35 and 25 feet in certain cases is of no avail as regards under bridges, or the abutments of over bridges, because of the liability to enlarge the bridges at a future time.

Bridge under Railway and over	Turnpike Road	Public Carriage Road.
	Feet.	Feet
Span, ...	35·00	25·00
Rise, ...	4·50	3·55
Clear headroom in centre,........................	16·50	15·55
Radius of intrados,..............................	36·28	23·00
Thickness of arch-ring,..........................	2·50	2·00
Depth of coating of puddle.......................	0·50	0·47
Depth of permanent way, say	2·00	2·00
Total height, roadway to rails,..................	21·5	20·00
To allow for additional depth in skew bridges, the above heights may be increased to	23·00	21·00
For *iron under bridges*, the above heights may be diminished to....................................	20·00	18·00

As to the *least dimensions of over bridges*, see Article 290, p. 426, for an example of the dimensions of the arch.

The clear headroom in the centre is usually,.....................	16·00 feet.
To this add, for the thickness of a stone arch,.	2·00
,, ,, ,, puddle coating, ...	0·50
,, ,, ,, roadway,....	1·00
Total,...................................	19·50
For an iron over bridge with flat girders the clear headroom may be reduced to about................................	14·50
Girders and roadway, say	2·50
Total..	17·00

The platform of an over bridge with iron girders may consist either of a series of transverse brick arches spanning across between the girders, which should be about 5 feet apart, and be held together by transverse ties sufficient to resist the thrust of the arches, or of cast iron plates with stiffening ribs above, covered with a layer of asphaltic concrete, or of buckled wrought iron plates, covered with a layer of asphaltic concrete. (See Article 375, p. 545.)

In crossing roadways in and near populous towns, where the ordinary dimensions of bridges would be too small, the width of the roadway, and, in the case of under bridges, the headroom, are usually fixed by agreement with the local authorities.

The *thickness of the abutments* of ordinary road bridges on lines of railway is usually from 1-5th to 1-6th of the span, and the counterforts are altogether of about one-third of the volume of the abutments. The wing walls are retaining walls, as to which, see Articles 265 to 268, pp. 401 to 407. In ordinary cases their thickness at the base is from 1-4th to 3-10ths of their height, and about one-half of that amount at the top, diminishing by steps or scarcements at the back of the wall; the face has a batter, of which 1 in 12 is an usual value.

Bridges over deep and wide cuttings may have three or five arches.

IV. *In selecting the line and levels*, the engineer should have regard to the crossings of existing lines of conveyance which may be required, bearing in mind that the earthwork of the approaches to those crossings, owing to its inconvenient situation, is more expensive than that of the railway itself. He should study to have as few bridges above the minimum size as possible; and with that view he should endeavour, as far as possible, to gain the necessary headroom partly by means of the elevation or depression of the railway above or below the existing road, and partly by means of an alteration of the level of that road.

The level occupied by existing lines of conveyance, if they have been well laid out, is usually the most favourable to economy of works; and for that reason there is generally an advantage in crossing such lines on the level, independently of the saving of the cost of bridges; but the choice between level crossings and bridges must be regulated mainly by considering whether the traffic is such as to make level crossings consistent with the public safety. In comparing the cost of a level crossing with that of a bridge, regard should be had to the necessity of having a gate keeper at the level crossing.

When the level of an existing road is to be lowered, special care must be taken that the cutting for that purpose can be properly drained.

The line of conveyance which causes the greatest impediment to

the passage of a railway is a canal . for it usually occupies precisely the most favourable level for economy of works; its level cannot in general be altered, nor can it be crossed on the level; and it can only be crossed *near* its own level by means of a swing bridge, which is inconvenient if the traffic is great.

In making a bridge over a canal, the span should be sufficient to contain the canal and its towing path without contracting their width; and care must be taken in founding the abutments not to disturb the canal. To prevent the escape of water it may be necessary to use coffer dams. (Article [illegible], p. 612.) The clear headroom is usually fixed by agreement; in ordinary cases it is 10 feet above the towing path, to be sufficient for a man on horseback.*

A passage under a canal may be made either by tunnelling at a sufficient depth, or by making a temporary or permanent diversion of the canal, and building an aqueduct bridge, which, if the diversion is to be temporary, will be on the original course of the canal, and if permanent, on that of the diversion. Canal and river bridges will be further considered in a later chapter.

437. **Ballast** is that portion of the PERMANENT WAY of a railway which forms a firm and dry foundation for the rails or for the sleepers by which they are supported. It is sometimes distinguished into *ballast* proper, or *under ballast*, which lies wholly below the sleepers or other supports of the rails, and *boxing*, or *upper ballast*, which is packed round the sleepers, chains, and rails, up to within two or three inches of the upper surface of the rails.

Examples of the breadth of the upper surface of the ballast have already been given in Article 435, pp. 656, 657. Its depth varies in different lines and according to the practice of different engineers; the following may be taken as its ordinary limits:—

		Feet.	Inches.		Feet.	Inches.
Lower ballast, ...	from	0	9	to	1	6
Upper ballast, or boxing,	„	0	6	to	0	9
Total depth, ...	„	1	3	to	2	3

In exploring the course of a projected railway, the engineer should give special attention to the sources from which good ballast can be obtained.

The best material is stone, broken as for road metal, into pieces not exceeding 6 ounces in weight (see Article 420, p. 627); but the stone does not need to be so hard as for roads; it is sufficient if it be of such hardness as would make it suitable for ordinary build-

* To admit of a horse rearing without danger to the rider, 12 feet of headroom should be allowed.

ing purposes. Stone that decays readily by the action of air and moisture ought to be carefully avoided. In the absence or scarcity of suitable stone, the slag of iron works, broken to a proper size, may be used; or alum-work refuse, which is shale burnt to the consistency of brick; or engine ashes. Next to broken stone and slag, as a material for ballast, is clean gravel; the lumps exceeding 6 ounces in weight being broken with the hammer. Clean sharp sand may be used, but it has the disadvantages, that in heavy falls of rain it may be washed away, and that in very dry weather it is blown into the air, and damages the rolling stock by lodging about the bearings of the axles and mechanism. In the absence of all other materials, pieces of clay suitable for bricks may be burnt until they are hard, and then broken down and used as ballast. As to the qualities of such clay, see Article 219, p. 363.

The labour of breaking and spreading a given quantity of ballast is about twice, or $2\frac{1}{2}$ times, that of excavating the same quantity of gravel.

438. **Sleepers** are pieces of material which rest on the ballast, as already stated (being firmly bedded on it by means of a beetle), and support the rails. At an early period in the history of railways, *stone blocks* were used for that purpose; they were placed at 3 feet apart from centre to centre, and measured, on horse-worked railways, about 18 inches × 12 inches × 9 inches, and on steam-worked railways, 2 feet × 2 feet × 1 foot; but they were found to form too hard and unyielding a base for traffic at high speeds, even with the aid of pieces of felt under the chairs, and their use was abandoned in favour of that of *timber sleepers*.

The best materials for timber sleepers are woods which withstand alternate wetness and dryness; and of those, the most generally employed in Europe is Larch. (Article 302, p. 443.) Various substances have been used for the preservation of timber sleepers: the most efficient is "creosote." (Article 311, p. 450, also pp. 799 and 805.)

Timber sleepers are either transverse or longitudinal. The former afford a ready and efficient means of preserving the gauge; the latter give the most equable and continuous support to the rails.

Transverse or *cross sleepers* are usually 9 to 10 feet long, from 9 to 10 inches broad, and from $4\frac{1}{2}$ to 5 inches deep. With the I-shaped section of rail resting on chairs bolted to the sleepers, and which is uniformly adopted on the railways in Britain, the sleepers are generally spaced about $2\frac{1}{2}$ feet apart.

When flat-bottomed rails are used of the type illustrated by Fig. 285, p. 667, a form in favour on American and Continental railways, the sleepers are placed rather closer together, so that there may be about fourteen sleepers in the length of a 10-yard long rail, whereas there would only be twelve sleepers under the

same length of the I-section rail. The flat-bottomed rail has the advantage of not requiring chairs, as it rests directly on the sleeper, and is spiked down to it. (See also p. 805.)

Besides timber sleepers, many different forms of iron and steel sleepers have been tried, principally on railways abroad, where, from various causes, the timber sleeper is liable to decay.

Longitudinal sleepers or *bearers* are usually from 12 to 14 inches broad, and from 6 to 7 inches deep, being made by sawing a square balk of timber in two. The rails may either have a continuous bearing on them, or may be supported by chairs at intervals of 30 inches or 3 feet. When the bearing is continuous it is usual to bolt or screw a plank of 7 or 8 inches wide and 1½ inch thick, or thereabouts, on the top of the sleeper, and upon this plank the base of the rail rests. In order to preserve the gauge, the pair of longitudinal bearers of a track of rails must be connected by means of cross-ties at intervals of about 5 or 6 yards. Special care should be taken that the ballast under the bearers is not impervious to water, lest they should confine water in the middle of the track.

Longitudinal and cross sleepers are sometimes combined, the cross sleepers being laid undermost. In this case the scantlings of the cross sleepers are made less than when cross sleepers are used alone, being usually about 7 feet × 7 inches × 3½ inches.

Cast iron sleepers are used of various forms. In Mr. Greaves's form the chair and sleeper are cast in one piece, the base being like an inverted bowl, near the summit of which are two holes, so that ballast can be put into the hollow and rammed from above. The gauge is preserved by transverse rods. In Mr. Samuel's form the rail is wedged with pieces of wood into a sort of cast iron trough with flat spreading wings. Another form is simply a flat oblong plate, with chairs cast on its upper side; and a fourth is an oblong trough wedged full of pieces of wood, on which the chairs rest. (See Clark *On Railway Machinery*.)

439. **Rails and Chairs.**—In the earlier days of the railway system rails formed of cast iron were used, but, with increased experience, wrought iron was substituted, as being more reliable due to its toughness than a crystalline substance like cast iron.

The rail lengths could also be obtained in greater length, thus necessitating fewer connections.

This wrought iron had to be carefully prepared, various kinds of bar iron being taken, piled together, re-heated, and rolled. This process had to be repeated several times to ensure toughness, and special care was given to the part which would ultimately be the top or running side of the rail when finished. This was attained by the introduction of charcoal iron, the object being

to have a surface upon which the wheels would roll with freedom and be at the same time durable.

This elaborate method is illustrated by fig. 280, which is an example of how the pile of bars was made up in order to be rolled into a rail. The introduction of steel, made by either the Bessemer or Siemens processes (see Appendix, p. 793), soon enabled manufacturers to turn out steel rails with much more rapidity than in the case of the wrought-iron rail, and now, as in other instances, steel-made rails have completely superseded those made from wrought iron. The form of steel rail in favour in this country has still the I-shaped figure in section which obtained with its wrought-iron predecessor (see p 667), and its weight per yard, and even the lengths themselves, vary with the requirements of the traffic of the various railway systems. Thus we have weights of 85, 90, 96, and even 100 lbs. per yard of rail, and lengths varying from 30 to 60 feet.

Fig. 280.

The sectional area of a rail, in square inches, is almost exactly one-tenth of the weight of one yard of its length in lbs.

On horse-worked railways, the weight of the rails per yard ranges from 28 lbs. to 25 lbs., the former weight being barely sufficient for durability. On the earlier of the high speed locomotive lines, a weight of about 60 lbs. to the yard was adopted; but the continual increase of the weight and speed of engines has rendered necessary a continual increase in the weight of rails; so that it now ranges from 70 to 100 lbs. per yard, or thereabouts.

As a general rule, it may be stated that the *weight of a yard of rail, if supported at intervals, should be 15 lbs. for each ton of the greatest load on one driving wheel.* When the bearing is continuous, about five-sixths of that weight is sufficient.

The *head* or top of a rail is usually about $2\frac{1}{4}$ inches broad, and has a very slight convexity in the middle, the radius of which is from 5 to 7 inches. In laying the rails they are carefully adjusted to the true gauge by means of a gauge-rod with shoulders on it at the proper distance apart. On straight lines the heads of the two rails should be exactly at the same level, and in curves they should be set to the proper "cant," as already explained in Article 434, p. 649. If cylindrical wheels are used for the rolling stock, the highest part of the head of each rail should be a tangent to a level line, or a line inclined at the proper cant, as the case may be; but if the wheels are tapered, the rails must be inclined inwards towards each other so as to be tangents at their highest points to the conical treads of the wheels. This inward inclination is given, where chairs are used, by casting the chairs so that their jaws may hold the rail in the proper position.

The figures adopted for the cross-sections of rails vary with the modes in which they are supported. A rail may be either

(1.) Supported on the base and stayed at the sides, or
(2.) Supported on a broad base alone, or
(3.) Hung by the shoulders;

and the bearings may be either

(A) at intervals (generally about 3 feet); or (B) continuous.

In figs. 281, 282, 283, and 284, the cross-section of the rail is of the I-shaped figure. When this rail was first introduced, it was intended that on the top becoming too much worn for further use, the rail should be turned upside down; but in general, by the time

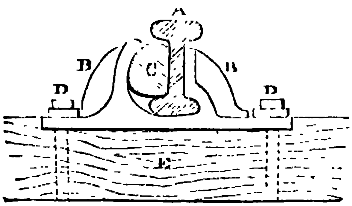


Fig. 281.

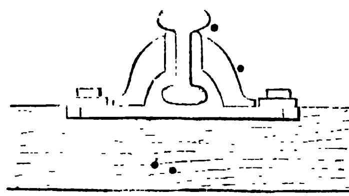


Fig. 282.

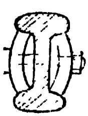


Fig. 283.

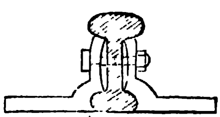


Fig. 284.

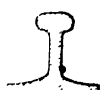


Fig. 285.

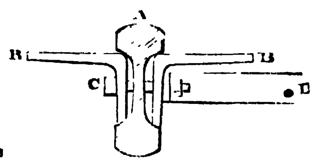


Fig. 286.

the top is worn out, the rail is unfit for further use. At present the bull-headed form is more generally used in this country than any other. The weight of rails of this form is from 85 to 96 lbs. per yard, and their depth between 5 and 6 inches. In fig. 281 the rail A is shown as supported by a common cast-iron chair B B, which is pinned by two compressed oak treenails D D. to cross sleeper E. The rail rests by its base or foot on the bottom of the chair, and is kept steady by being firmly held between the inner jaw of the chair and a compressed oak wedge or key, C, which is driven between the rail and the outer jaw. (See p. 794.)

The inner faces of the jaws or cheeks of the chair are chilled. (See Article 352, p. 499.) An ordinary chair weighs about as much as *one foot* of the rail which it is intended to support; a *joint-*

chair, for supporting the ends of two adjoining rails, is from one-third to one-half heavier, and its outer end is usually fastened down by two pins instead of one. Fig. 282 represents a sort of joint-chair, introduced by Mr. William Johnstone, which has been found to answer well; it is slid on to the rails, which its jaws fit exactly.

It has now, however, become very generally the practice to connect adjoining rails by the *fish-joint;* the ends of the rails being supported by a pair of ordinary chairs, which they overhang by 12 or 15 inches, and being united to each other by a pair of *fish-pieces,* about 18 or 20 inches long, bolted together through the rails by four bolts. Fig. 283 shows in cross-section the figure of which the fish-pieces are made in order that they may abut at their upper and lower edges against the head and foot of each rail, and thus be wedged into their places. The bolt-holes in the rails are made slightly oblong horizontally, to allow for changes of length by heat and cold, which may amount, in ordinary cases, to about 1-2000th or 1-2500th of the length of each rail (p. 527).

Fig. 284 is the *bracket fish-joint,* in which the fish-pieces are of angle iron, and answer the purpose of a joint-chair; their horizontal bases being bolted down to a pair of cross sleepers.

Among rails which are supported *on a broad base without chairs* may be mentioned the *foot rail,* fig. 285, which is fastened down by means of fang-bolts to longitudinal or cross sleepers; the *bridge rail* (fig. 229, p. 518), and the *Barlow rail* (fig. 230, p. 518).

Bridge rails are made from 3 to 5 inches deep, and from 7 to 6 inches in breadth of base, and are bolted to the sleepers, either through slightly oblong holes, or by fang-bolts holding down the edges of the base. When supported on a continuous bearing, they weigh about 65 lbs. per yard; when supported at intervals on cross sleepers, 82 lbs. per yard. In the latter case each joint is secured by holding the bases of the two rails between a pair of cast iron jaws, drawn together by transverse bolts which pass under the rails. (This system was introduced by Sir John Macneill, and is used on Irish lines).

The Barlow rail is now made with a portion of each wing flat and horizontal, so that it approaches somewhat nearer to the bridge shape than fig. 230 shows. It is about a foot broad, 5 or 6 inches deep over all, and weighs from 90 to 100 lbs. per yard. It rests directly on the ballast, without sleepers or chairs, and the gauge is preserved by means of cross-ties of angle iron. The joint is in fact a sort of fish-joint, the ends of the rails being connected together by being bolted through oblong holes to a *saddle-piece,* 3 feet long, which is a bar somewhat resembling the rails in cross-section, and made exactly to fit the hollow at the under side of the rail.

The system of supporting rails *by the shoulders* of the enlarged head, which is the most favourable of all to steadiness, was practised more than twenty years ago, with the shallow T-shaped rails then used for horse-worked railways, by Mr. David Rankine. The flat-bottomed type of rail, shown in fig 285, is used in America and on the Continent of Europe; they weigh about 85 lbs. to the yard, and are spiked down to the sleepers.

Fig. 286 represents Mr. Adams's *suspended girder* rail, in which the vertical web, not having to sustain compression, is made thinner and deeper than in ordinary rails, so that, for example, a rail of 75 lbs. to the yard is 7 inches deep. The rail A has a continuous bearing at the shoulders upon a pair of angle iron brackets, B, B, whose lower edges press against the foot of the rail, so that they are wedged into the hollow sides of the rail by bolts, C, about 3 feet apart, passing through oblong holes. D is a cross tie-bar, to preserve the gauge. The total breadth across the wings of the brackets ranges from 9 to 14 inches, according to the weight of the traffic; and those wings rest directly on the ballast. The rails and the brackets are laid so as to break joint. Another mode of constructing this sort of permanent way is to substitute pieces of creosoted timber, about 5 inches square, for the angle-iron brackets.*

440. **Rails for Level Crossings of Roads.**—When the covering of a road is of broken stone, the ordinary permanent way of the railway may be laid across it, with the heads of the rails rising about $\frac{3}{4}$ of an inch above the road metal. Switches, points, and crossings of rails should not occur on a level crossing of a road, nor should any rails on such a crossing approach nearer to each other than about 6 inches, lest horses should be injured or disabled by their feet being wedged between rails, or the caulkers of their shoes, used in frosty weather, getting jammed in the openings of points, switches, and rail crossings.

Crossings of paved roads also may be made with the ordinary rails, grooves for the flanges of the wheels being cut in the paving-stones alongside of the rails.

Rails of a special form are sometimes used for level crossings of roads. They are usually H-formed, the upper side presenting a groove about $2\frac{1}{2}$ inches broad and $1\frac{1}{2}$ inch deep, between two flanges, the outer of which, being the head of the rail on which the wheel runs, must be $2\frac{1}{2}$ inches broad; while the inner flange may be of the same breadth or of a less breadth, according as the rail is to be reversible or not.

441. **Junctions and Connections of Lines of Rails—Traversers—Turntables.**—I. The *junction* of two lines of rails is effected either by means of a pair of those tapering moveable rails called *switches*,

* See Addendum, p. 794.

connected together and worked by the same handle, or by means of a switch at the side *from* which a carriage leaving the main line turns, and a fixed *point* at the other side. The former arrangement is considered the safer where the speed of the traffic is great.

On a narrow gauge line, the ordinary distance between the tip of the switch and the *crossing*, where the two tracks finally become clear of each other, is about 80 feet, so that if one of the tracks is straight the other has a curvature of about 640 feet radius.

Switches are made self-acting by a weight which pulls them into that position which suits the main stream of traffic, so that they require to be held by force in the contrary position. The handles by which switches are worked are so shaped and painted that their position can be distinctly seen from a considerable distance.

It was formerly the practice to notch the sides of the fixed rails so as to receive the tips of the switches; but this has been rendered unnecessary by the introduction of an improved form of switch.*

As far as possible, switches on the main tracks of a line of railway should point in the ordinary direction of the traffic; *facing-points*, as those which point in the contrary direction are called, should only be used in cases of necessity, and then with precautions sufficient to obviate the risk of a train being accidentally turned on to a wrong line.

II. A *connection* between two parallel lines of rails is usually made, where there is room enough, by means of an oblique line of rails with switches at each end. On the narrow gauge, the length of such a connection is about 180 feet.

III. A *traverser* affords the most convenient mode of shifting carriages between parallel lines of rails at a terminal station, where there is not room enough for an ordinary connection. It is a platform supporting a line of rails, long enough for a carriage to stand upon, and supported on wheels which roll on a transverse line of rails at a lower level.

IV. *Turntables* serve to connect lines of rails which cross each other at right angles, or which radiate from a central point. A turntable consists essentially of the following parts:—A foundation of masonry or concrete, a circular cast iron base, having a pivot in the centre, and a race or track for rollers round the circumference; a set of conical rollers, carried in a frame which turns about the pivot; a deck or platform, supported on the pivot at its centre and on the rollers at its circumference, carrying one or more lines of rails, and provided with one or more catches to fix it in different positions. The greater the proportion of the weight borne by the pivot, and the less that borne by the rollers, the less is the friction.

* First introduced by Messrs. Ransome and May.

A turntable which floats in a cylindrical water-tank was invented by Mr. Adams.

Carriage turntables are usually 12 or 14 feet in diameter, and carry two lines of rails at right angles to each other.

A turntable for an engine and tender is 40 feet in diameter, or thereabouts; it usually carries but one line of rails, and is turned round by the aid of wheelwork. Such turntables are required at stations, to reverse the engines, independently of the connection of lines of rails. Turntables for engine sheds are occasionally made with two parallel lines of rails.

For details as to the construction of these and other railway fittings, see Mr. D. K. Clark's work *On Railway Machinery*.

442. **Stations.**—The best positions for stations, in a purely engineering point of view, and the manner in which they affect questions of curves and gradients, have already been discussed in Article 435, p. 658. It may here be added that the engineer should specially attend to the means of draining the stations, of supplying them with good water, and of getting access to them by roadways, for the arrival and departure of passengers and goods. Passenger platforms are from 2 to 3 feet above the level of the rails, and are best made of strong flags resting on longitudinal walls; at their ends, they should descend gradually to the level of the rails by *ramps* or slopes of about 1 in 10, rather than by flights of steps. They should be, according to Mr. Clark, at least 20 feet broad when used at one side, 30 or 40 when used at both sides; but they are often made of much smaller breadths. The roof should, if possible, be in one span over the whole station; if intermediate pillars are used, they should be placed in the middle of broad platforms, and not near lines of rails if it can be avoided; they should never, on any account, be nearer a line of rails than 4 feet. Care should be taken that sheds have proper means of ventilation. The extent and arrangement of the station and its fittings, as affected by questions of the amount of traffic and the best means of accommodating it, are foreign to the subject of the present work. For examples of stations, see Mr. Clark's article "On Railways" in the *Encyc. Brit.*

442 A. **Pipe-Culverts—Mile-Posts—Gradient-Posts—Telegraph.**—The construction of culverts large enough to be accessible for purposes of repair, in order to carry gas-pipes and water-pipes under a railway, is enjoined by law in Britain; as is also the erection of numbered mile-posts at every quarter of a mile. Gradient-posts, at changes of gradient, having boards to show the direction and rate of inclination, are useful to the engine drivers. Where trains run at very short intervals, a line of electric telegraph may be considered as almost essential to the safe working of the railway. (See p. 706.)

CHAPTER II.

OF THE COLLECTION, CONVEYANCE, AND DISTRIBUTION OF WATER.

SECTION I.—*Theory of the Flow of Water, or of Hydraulics.*

443. **Pressure of Water—Head.**—The laws of the pressure of a mass of water, when at rest, against any surface which it touches, have already been explained in Article 107, p. 164.

In all questions of hydraulics, it is convenient to express the intensity of the pressure of water in *feet of water;* that is, in terms of the intensity of the pressure of a column of water one foot high upon its base, as an unit. A pressure so expressed is sometimes called a *head of pressure.* In p. 161 two values of that unit, for pure water at the temperatures of 39°·1 and 62° respectively, have been compared with other units; in the following table a comparison of the same sort is given in greater detail, the heaviness assigned to pure water being 62·4 lbs. per cubic foot, which is almost perfectly exact at a temperature of about 52°·3 Fahrenheit, and near enough to the truth for practical purposes at other temperatures, and is also a convenient value for calculation:—

COMPARISON OF HEADS OF WATER IN FEET, WITH PRESSURES IN VARIOUS UNITS.

One foot of water at 52°·3 Fahr. =	62·4	lbs. on the square foot.
" "	0·4333	lb. on the square inch.
" "	0·0295	atmosphere.
" "	0·8823	inch of mercury at 32°.
" "	773·	feet of air at 32°, and one atmosphere.
One lb. on the square foot, ...	0·016026	foot of water.
One lb. on the square inch, ...	2·306	feet of water.
One atmosphere of 29·922 inches of mercury, ...	33·9	" "
One inch of mercury at 32°, ...	1·1334	" "
One foot of air at 32°, and one atmosphere, ...	0·001294	" "
One foot of average sea water, ...	1·026	foot of pure water.

The TOTAL HEAD of a given particle of water is found by adding together the following quantities:—

The *head of pressure*, or intensity of the pressure exerted by the particle, expressed in feet of water.
The *head of elevation*, or actual height of the particle above some fixed or "*datum*" level.

In stating the pressure or head of a particle of water it is usual *not* to include the *atmospheric pressure*, so that the *absolute* or true pressure exceeds the pressure as stated in the customary way by one atmosphere. When the absolute pressure is exactly one atmosphere, the pressure as stated in the customary way is *nothing;* when the absolute falls short of the atmospheric pressure by so many lbs. on the square inch, or so many feet of water, the customary mode of stating that fact is to say that there are so many lbs. on the square inch, or so many feet, of *vacuum*.

The atmospheric pressure, at the level of the sea, varies from about 32 to 35 feet of water, and diminishes at the rate nearly of 1-100th part of itself for each 262 feet of elevation above that level.

444. **Volume and Mean Velocity of Flow.**—The *volume of flow* or *discharge* of a stream of water is expressed in units of volume per unit of time.

The most convenient unit of volume is the *cubic foot;* but in calculations relating to the water supply of towns it is customary to use the *gallon*.

The following is the relation between those units:—

One gallon = 0·1604 cubic foot (being 10 lbs. of water); and
One cubic foot = 6·2355 gallons;

but in ordinary calculations respecting water-works it is sufficiently accurate to make one gallon = 0·16 cubic foot, and one cubic foot = $6\frac{1}{4}$ gallons.

Of different *units of time*, the *second* is the most convenient in mechanical calculations; the *minute* is the customary unit in stating the discharge of streams; the *hour*, the *day*, and longer periods are used in calculations as to drainage and water supply.

The variety of *units of discharge* is thus very great. The *cubic foot per second* is the most convenient in mechanical calculations.

The *mean velocity* of a stream at a given cross-section is found by dividing the discharge, or volume of flow, by the area of the cross-section, and is most conveniently expressed in feet per second.

445. **Greatest and Least Velocities.**—Inasmuch as every stream of fluid that flows in a channel is retarded by friction against the

material of the channel, the velocity of the fluid particles is different at different points of the same cross-section, being greatest in the centre and least at the border. In open channels, like those of rivers, the ratio of the mean velocity to the greatest or central velocity is given approximately by the following formula of Prony:—

$$\frac{\text{mean velocity}}{\text{greatest velocity}} = \frac{\text{greatest velocity} + 7 \cdot 71 \text{ feet per second}}{\text{greatest velocity} + 10 \cdot 28 \text{ feet per second}}. \quad (1.)$$

The least velocity, or that of the particles in contact with the bed, is about as much less than the mean velocity as the greatest velocity is greater than the mean. In ordinary cases, the least, mean, and greatest velocities may be taken as bearing to each other nearly the proportions of 3, 4, and 5. In very slow currents they are nearly as 2, 3, and 4. (See p. 792.)

446. **General Principles of Steady Flow.**—The *steady* motion of a mass of fluid, as distinguished from unsteady motion, means that kind of motion in which the velocity and direction of motion of a particle depend on its *position* alone, and not jointly on position and time; so that each particle of the series of particles which successively come to a given point, assumes a certain velocity and direction of motion proper to that point. It is, in short, the motion of a *permanent current*, as distinguished from that of a varying current, or that of a wave.

In order to acquire velocity from a state of rest, or an increase of velocity, a fluid particle must pass from a place of *greater total head* to a place of *less total head*. This it may do either by actual descent from a higher to a lower level, or by passing from a place of more intense pressure to a place of less intense pressure, or by both those changes combined. The *loss of head* thus incurred is connected with the velocity produced by the following laws:—

I. In a liquid without friction the loss of head in producing a given increase of velocity is equal to the height of vertical fall which would produce the same increase of velocity in a body falling freely; in other words, the loss of head is equal to the *height due to the acceleration*; and if the particle starts from a state of rest, that height is called the *height due to the velocity*, and is given by the following formula, where v is the velocity in feet per second:—

$$\text{height in feet} = v^2 \div 64 \cdot 4. \ldots\ldots\ldots\ldots\ldots\ldots (1.)$$

II. If the motion of the liquid is impeded by friction, there is an additional loss of head, bearing to the height due to the velocity of flow a certain proportion, depending on the figure and dimensions

of the channel and openings traversed by the stream, and other circumstances.

The combination of those two principles may be thus expressed: Let h denote the *loss of head*, in feet; then

$$h = (1 + F) \frac{v^2}{64 \cdot 4}; \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.)$$

in which F is a factor, determined by experiment, expressing the proportion which the loss of head by friction bears to the height due to the velocity.

The inverse formula, for finding the velocity from the loss of head, is as follows:-

$$v = 8 \cdot 025 \sqrt{\frac{h}{1 + F}} \quad .(3.)$$

The velocity computed from a given height, on the supposition that there is no friction, by the formula $v = 8 \cdot 025 \sqrt{h}$, is sometimes called the "theoretical velocity."

In an open channel the loss of head h consists wholly in diminution of the "head of elevation," and is the *actual fall* of the upper surface of the stream. In a close pipe it may consist wholly or partly of diminution of the "head of pressure," and is then called *virtual fall*. To express this in symbols,

Let z_1 denote the elevation above a fixed datum, and

p_1, the head of pressure at a point in the reservoir from which a pipe is supplied, the velocity at that point being insensible, so that

$z_1 + p_1$ is the *total head in still water;* also let

z denote the elevation above the datum, and

p, the head of pressure at a given point in the pipe, at which the loss of head, as computed by equation 2, is h; then the total head at this point is,

$$z + p = z_1 + p_1 - h; \quad \ldots\ldots\ldots\ldots\ldots\ldots (4.)$$

and the pressure, in feet of water, is

$$p = z_1 + p_1 - z - h. \quad \ldots\ldots\ldots\ldots\ldots\ldots (5.)$$

The pressure of flowing water, as thus diminished by loss of head, is called HYDRAULIC PRESSURE, to distinguish it from the pressure of still water, called *hydrostatic pressure.*

In an open channel, equation 5 is simplified by the fact that for the upper surface of the stream, and all surfaces parallel to it, h is simply $= z_1 - z$; so that $p = p_1$, if the two points are at equal depths below the surface.

If the water has a sensible velocity of flow *at the starting point*, the loss of head required is diminished to the extent of the height due to that *velocity of approach*, as it is called. Thus, let v_0 be the velocity of approach; then, instead of equation 2, we must use the following

$$h = (1 + \mathrm{F}) \frac{v^2}{64 \cdot 4} - \frac{v_0^2}{64 \cdot 4}; \qquad (6.)$$

and if v_0 bears *a known ratio* to v, let that ratio be $v_0 \div v = r$; then the above equation becomes,

$$h = (1 + \mathrm{F} - r^2) \frac{v^2}{64 \cdot 4}; \ldots\ldots\ldots\ldots\ldots\ldots (7.)$$

which gives, for the inverse formula,

$$v = 8 \cdot 025 \sqrt{\frac{h}{1 + \mathrm{F} - r^2}} \ldots\ldots\ldots\ldots\ldots\ldots (8.)$$

When a stream flows with an *uniform speed* down an *uniform channel*, and two cross-sections of that channel are compared together, the velocities v_0 and v are equal, and $r = 1$; in this case, the *whole* loss of head between the two cross-sections is expended in overcoming friction; and equations 7 and 8 are reduced to the following:—

$$h = \mathrm{F}\, v^2 \div 64 \cdot 4; \ldots\ldots\ldots\ldots\ldots\ldots (9.)$$

$$v = 8 \cdot 025 \sqrt{h \div \mathrm{F}} \ldots\ldots\ldots\ldots\ldots (10.)$$

The following table gives examples of heights in feet due to velocities in feet per second, as computed by equation 1. It is exact for latitude $54^\circ \frac{3}{4}$, and near enough to exactness for practical purposes in all latitudes. The most convenient table, however, for calculating either heights from velocities or velocities from heights is an ordinary table of squares and square roots:—

v	h	v	h	v	h	v	h	v	h
1	·01553	17	4·4876	32·2	16·100	48	35·776	76	89·688
2	·06211	18	5·0311	33	16·910	49	[illegible]·283	78	94·472
3	·13975	19	5·6050	34	17·950	50	38·820	80	99·379
4	·24845	20	6·2112	35	19·022	52	41·987	82	104·41
5	·38820	21	6·8478	36	20·124	54	45·280	84	109·56
6	·55901	22	7·5155	37	21·257	56	48·695	86	114·84
7	·76087	23	8·2143	38	22·422	58	52·235	88	120·25
8	·99379	24	8·9441	39	23·618	60	55·901	90	125·78
9	1·2578	25	9·7050	40	24·845	62	59·688	92	131·43
10	1·5528	26	10·497	41	26·102	64	63·602	94	137·20
11	1·8789	27	11·320	42	27·391	64·4	64·400	96	143·10
12	2·2360	28	12·174	43	28·711	66	67·640	98	149·13
13	2·6242	29	13·059	44	30·062	68	71·800	100	155·28
14	3·0435	30	13·975	45	31·444	70	76·087		
15	3·4938	31	14·922	46	32·857	72	80·496		
16	3·9752	32	15·901	47	34·301	74	85·029		

(See p. 804.)

447. Friction of Water.—The following are the values of the factor of friction F in the formulæ of Article 446, as ascertained by experiment, for the cases of most common occurrence in practice.

I. *Friction of an orifice in a thin plate*—

$$F = 0.054. \quad \ldots\ldots\ldots\ldots\ldots\ldots (1.)$$

II. *Friction of mouthpieces or entrances from reservoirs into pipes.*—Straight cylindrical mouthpiece, perpendicular to side of reservoir—

$$F = 0.505. \quad \ldots\ldots\ldots\ldots\ldots\ldots (2.)$$

The same mouthpiece making the angle θ with a perpendicular to the side of the reservoir—

$$F = 0.505 + 0.303 \sin \theta + 0.226 \sin^2 \theta. \ldots\ldots\ldots (3.)$$

For a mouthpiece of the form of the "contracted vein," that is, one somewhat bell-shaped, and so proportioned that if d be its diameter on leaving the reservoir, then at a distance $d \div 2$ from the side of the reservoir it contracts to the diameter $\cdot 7854\, d$,—the resistance is insensible, and F nearly $= 0$.

III. *Friction at sudden enlargements.*—Let A_1 be the sectional area of a channel, discharging Q cubic feet of water per second, in which a sluice, or slide valve, or some such object, produces a sudden contraction to the smaller area a, followed by a sudden enlargement to the area A_2. Let $v = Q \div A_2$ be the velocity in the second enlarged part of the channel. The *effective* area of the orifice a will be $c\,a$, c being a *co-efficient of contraction* of the stream flowing through it, whose value may be taken at $\cdot 618 \div \sqrt{1 - \cdot 618 \frac{a^2}{A_1^2}}$. Let the ratio in which the effective area of the channel is suddenly enlarged be denoted by

$$r = A_2 \div c\,a = \frac{A_2}{a} \sqrt{\left(2 \cdot 618 - 1 \cdot 618 \frac{a^2}{A_1^2}\right)}. \ldots\ldots (4.)$$

Then $r\,v$ is the velocity in the most contracted part. It appears that all the energy due to the *difference* of the velocities, $r\,v$ and v, is expended in fluid friction, and consequently that there is a loss of head given by the formula—

$$(r - 1)^2 \cdot \frac{v^2}{2\,g}; \quad \ldots\ldots\ldots\ldots\ldots\ldots (5.)$$

so that in this case

$$F = (r - 1)^2. \quad \ldots\ldots\ldots\ldots\ldots\ldots (6.)$$

IV. *Friction in pipes and conduits.*—Let A be the sectional area

of a channel; b its *border*, that is, the length of that part of its girth which is in contact with the water; l the length of the channel, so that $l\,b$ is the frictional surface; and for brevity's sake let $A \div b = m$; then, for the friction between the water and the sides of the channel—

$$F = f \cdot \frac{l\,b}{A} = \frac{f\,l}{m}; \quad \text{(7.)}$$

in which the co-efficient f has the following values:—

For iron pipes (Darcy), ... $f = 0{\cdot}005 \left(1 + \frac{1}{48\,m\,(\text{feet})}\right)$; (8.)

For open conduits (Weisbach), $f = 0{\cdot}0074 + \frac{0{\cdot}00023}{v}$. ....(9.)

The quantity $m = A \div b$ is called the "*hydraulic mean depth*" of channel, and for cylindrical and square pipes running full is obviously *one-fourth* of the diameter; and the same is its value for a semicylindrical open conduit, and for an open conduit whose sides are tangents to a semicircle of a diameter equal to twice the greatest depth of the conduit.

In an open conduit, the loss of head,

$$h = \frac{f\,l}{m} \cdot \frac{v^2}{2\,g}, \quad \text{(10.)}$$

takes place as an actual fall in the surface of the water, producing a declivity at the rate

$$i = \frac{h}{l} = \frac{f}{m} \cdot \frac{v^2}{2\,g}; \quad \text{(11.)}$$

and by the last two formulæ are to be determined the fall and the rate of declivity of open channels which are to convey a given flow. In close pipes, the loss of head takes place in the total head; and the ratio $i = h \div l$ is called the *virtual declivity*.

V. For *bends in circular pipes*, let d be the diameter of the pipe, ρ the radius of curvature of its centre line at the bend, θ the angle through which it is bent, π two right angles; then, according to Professor Weisbach,

$$F = \frac{\theta}{\pi} \left\{ 0{\cdot}131 + 1{\cdot}847 \left(\frac{d}{2\,\rho}\right)^{\frac{7}{2}} \right\} \quad \text{(12.)}$$

VI. For *bends in rectangular pipes*,

$$F = \frac{\theta}{\pi} \left\{ 0{\cdot}124 + 3{\cdot}104 \left(\frac{d}{2\,\rho}\right)^{\frac{7}{2}} \right\} \quad \text{(13.)}$$

VII. For *knees*, or sharp turns in pipes, let θ be the angle made by the two portions of the pipe at the knee; then

$$F = 0{\cdot}946 \sin^2 \frac{\theta}{2} + 2{\cdot}05 \sin^4 \frac{\theta}{2}. \quad \ldots\ldots\ldots\ldots (14.)$$

VIII. *Summary of losses of head.*—When several successive causes of resistance occur in the course of one stream, the losses of head arising from them are to be added together; and this process may be extended to cases in which the velocity varies in different parts of the channel, in the following manner:—

Let the final velocity at the cross section, where the loss of head is required, be denoted by v;

Let the ratios borne to that velocity by the velocities in other parts of the channel be known; $r_0 v$ being the "velocity of approach" (Article 446, p. 676), $r_1 v$ the velocity in the first division of the channel, $r_2 v$ in the second, and so on; and let F_1 be the sum of all the factors of resistance for the first division, F_2 for the second, and so on; then the loss of head will be—

$$h = \frac{v^2}{64{\cdot}4}\left(1 - r_0^2 + F_1 r_1^2 + F_2 r_2^2 + \&c.\right);$$

an expression which may be abbreviated into the following: } (15.)

$$h = \frac{v^2}{64{\cdot}4}\left(1 - r_0^2 + \Sigma \cdot F r^2\right).$$

448. **Contraction of Stream from Orifice—Co-efficients of Discharge.**—The fact of the contraction of a jet or stream that flows from an orifice has already been referred to. It is caused by the convergence of the particles towards the orifice before they pass through it, which convergence continues for a time after the particles pass the orifice. The result is, that the *effective* area of the orifice, or area of the "*contracted vein*," which is to be used in computing the discharge, is less than the total area in a proportion which is called the *co-efficient of contraction*.

Sometimes it is impossible to distinguish between the effect of friction in diminishing the velocity (expressed by $1 \div \sqrt{1 + F}$), and that of contraction in diminishing the area of the stream. In such cases the ratio in which the actual discharge is less than the product of the "theoretical velocity" (Article 446, p. 675) and the total area of the orifice, is called the *co-efficient of efflux* or *of discharge*.

The quantities given in the following statements and tables are some of them real co-efficients of contraction, and some co-efficients of discharge. In hydraulic formulæ, such co-efficients are usually denoted by the symbol c.

In sharp-edged orifices the friction is almost inappreciable (see Article 447, Case I.); in those with flat or rounded borders its effects become sensible, and in tubes or other channels of such length as to guide all the particles along their sides there is no contraction, and friction operates alone in diminishing the discharge.

In all the *sharp-edged orifices* here mentioned the edge is supposed to be formed at the *inner* or up-stream side of the plate by chamfering or bevelling the outer side. Were the inner side of the plate chamfered, it would guide the stream, and alter the contraction to an uncertain amount.

I. *Sharp-edged circular orifices in flat plates*; $c = \cdot 618$....(1.)

II. *Sharp-edged rectangular orifices in vertical flat plates.*—In this case the co-efficient depends partly on the proportions of the dimensions of the orifice to each other, and partly on the proportion borne by the breadth of the orifice to the *charge* or head. The co-efficient is intended to be used in the following formula for the discharge in cubic feet per second, A being the area of the orifice in square feet; and h the head, measured from the *centre* of the orifice to the *level of still water*.

$$Q = 8{\cdot}025\, c\, A \sqrt{h} \text{ ....(2.)}$$

The co-efficients are given on the authority of experiments of Poncelet and Lesbros on orifices about 8 inches wide. They have not been reduced to a general formula.

CO-EFFICIENTS OF DISCHARGE FOR RECTANGULAR ORIFICES.

Head ÷ Breadth.	Height of Orifice ÷ Breadth. 1	0·5	0·25	0·15	0·1	0·05
						·709
0·10	...	...	...	...	·660	·698
0·15	...	...	...	·638	·660	·691
0·20	...	...	·612	·640	·659	·685
0·25	...	...	·617	·640	·659	·682
0·30	...	·590	·622	·640	·658	·678
0·40	...	·600	·626	·639	·657	·671
0·50	...	·605	·628	·638	·655	·667
0·60	·572	·609	·630	·637	·654	·664
0·75	·585	·611	·632	·635	·653	·660
1·00	·592	·613	·634	·634	·650	·655
1·50	·598	·616	·632	·632	·645	·650
2·00	·600	·617	·631	·631	·642	·647
2·50	·602	·617	·631	·630	·640	·643
3·50	·604	·616	·629	·629	·637	·638
4·00	·605	·615	·627	·627	·632	·627
6·00	·604	·613	·623	·623	·625	·621
8·00	·602	·611	·619	·619	·618	·616
10·00	·601	·607	·613	·613	·613	·613
15·00	·601	·603	·606	·607	·608	·609

The co-efficients in the preceding table include a correction for the error occasioned by measuring the head from the *centre* of the orifice instead of from the point where the mean velocity occurs, which is somewhat above the centre. That correction is inappreciable when the head exceeds 3 times the height of the orifice.

III. *Sharp-edged rectangular notches* (or orifices extending up to the surface) *in flat vertical weir boards.*—The area of the orifice is measured up to the *level of still water* in the pond behind the weir.

Let b = breadth of the notch;
B = total breadth of the weir; then

$$c = \cdot 57 + \frac{b}{10\,B}; \qquad \text{(3.)}$$

provided b is not less than $B \div 4$.

IV. *Sharp-edged triangular or V-shaped notches in flat vertical weir boards* (from experiments by Professor James Thomson.)—Area measured up to the level of still water.

$$\left.\begin{array}{l} \text{Breadth of notch} = \text{depth} \times 2;\ c = \cdot 595; \\ \text{Breadth of notch} = \text{depth} \times 4;\ c = \cdot 620 \end{array}\right\} \qquad \text{(4.)}$$

V. *Partially-contracted sharp-edged orifice.* (That is to say, an orifice towards part of the edge of which the water is guided in a direct course, owing to the border of the channel of approach partly coinciding with the edge of the orifice).

Let c be the ordinary co-efficient;
n, the fraction of the edge of the orifice which coincides with the border of the channel;
c', the modified co-efficient; then

$$c' = c + \cdot 09\,n. \qquad \text{(5.)}$$

VI. *Flat or round-topped weir,* area measured up to the level of still water—

$$c = \cdot 5 \text{ nearly}. \qquad \text{(6.)}$$

VII. *Sluice in a rectangular channel*—

$$\left.\begin{array}{r} \text{vertical};\ c = 0{\cdot}7; \\ \text{Inclined backwards to the horizon at } 60^\circ;\ c = 0{\cdot}74; \\ \text{,,}\qquad\text{,,}\qquad\text{,,}\qquad \text{at } 45^\circ;\ c = 0{\cdot}8. \end{array}\right\} \text{(7.)}$$

VIII. *Incomplete contraction;* see Article 477, Division III., p. 677.

449. **Discharge from Vertical Orifices, Notches, and Sluices.**—When the height of an orifice in the vertical side of a reservoir

does not exceed about one-half or one-third of its depth below the surface, the head measured from the centre of the orifice to the level of still water may be used, without sensible error, to compute the mean velocity of a flow, and the discharge; so that the formula for the discharge is

$$Q = 8{\cdot}025\ c\ A \sqrt{h}; \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1.)$$

A being the total area of the orifice, and c the proper co-efficient of contraction.

When the height of the orifice exceeds about one-half of the head of water, and especially when the orifice is a *notch* extending to the surface, it is not sufficiently accurate to measure the head simply from the level of still water to the centre of the orifice; but the area of the orifice is to be conceived as divided into a number of horizontal bands, the area of each such band multiplied by the velocity due to its depth below the surface of still water, the products summed or integrated, and the sum or integral multiplied by a suitable co-efficient of contraction.

To express this in symbols, let b be the breadth, $d\,h$ the height of one of the horizontal bands, so that $b\ d\ h$ is its area; h, the depth of its centre below the level of the surface of still water in the reservoir; h_0, the depth of the upper edge of the orifice, and h_1 that of its lower edge, below the same level; c, the co-efficient of contraction; Q, the discharge in cubic feet per second; then

$$Q = 8{\cdot}025\ c \int_{h_0}^{h_1} b \sqrt{h} \cdot d\,h. \ldots\ldots\ldots\ldots\ldots (2.)$$

For co-efficients of contraction, see Article 448.

The following are the most important cases:—

I. *Rectangular orifice*; $b =$ constant.

$$Q = 8{\cdot}025\ c \times \frac{2}{3}\, b \left(h_1^{\frac{3}{2}} - h_0^{\frac{3}{2}}\right) = 5{\cdot}35\ c\ b \left(h_1^{\frac{3}{2}} - h_0^{\frac{3}{2}}\right). \ (3.)$$

It is seldom necessary to use this formula in practice; for the co-efficients in the table by Poncelet and Lesbros (see p. 680) comprehend, as has been stated, the correction for the error arising from using the head at the centre of the orifice simply, as in equation 1.

II. *Rectangular notch, with a still pond*; $b =$ constant, $h_0 = 0$; h_1 measured from the lower edge of the notch to the level of still water.

$$\left.\begin{aligned} Q &= 8{\cdot}025\ c \times \frac{2}{3}\, b\, h_1^{\frac{3}{2}} = 5{\cdot}35\ c\ b\, h_1^{\frac{3}{2}} \\ &= \left(3{\cdot}05 + {\cdot}535\, \frac{b}{B}\right) b\, h_1^{\frac{3}{2}} \end{aligned}\right\} \ldots\ldots\ldots (4.)$$

The last expression is founded on the formula for the co-efficient c, given in Article 448, Division III., p. 681, B being the whole breadth of the weir.

TABLE OF VALUES OF c AND $5·35\,c$.

$\frac{b}{B}$,.....	1·0	0·9	0·8	0·7	0·6	0·5	0·4	0·3	0·25
c,......	·67	·66	·65	·64	·63	·62	·61	·60	·595
$5·35\,c$,	3·58	3·53	3·48	3·42	3·37	3·32	3·26	3·21	3·18

The *cube of the square root of the head*, $h_1^{\frac{3}{2}}$, is easily computed as follows, by the aid of an ordinary table of squares and cubes: look in the column of squares for the nearest square to h_1; then opposite, in the column of cubes, will be an approximate value of $h_1^{\frac{3}{2}}$.

III. *Rectangular notch, with current approaching it.*—When still water cannot be found, to measure the head h_1 up to, let v_0 denote the velocity of the current at the point up to which the head is measured, or *velocity of approach:* compute the height due to that velocity as follows:—

$$h_0 = v_0^2 \div 64·4;$$

then the flow is the difference between that from a still pond due to the height $h_1 + h_0$, and that due to the height h_0; so that it is given by the formula

$$Q = 5·35\,c\,b\,\{(h_1 + h_0)^{\frac{3}{2}} - h_0^{\frac{3}{2}}\}\,. \ldots\ldots\ldots\ldots\ldots (5.)$$

When v_0 cannot be directly measured, it can be computed approximately by taking an approximate value of Q from equation 4, and dividing by the sectional area of the channel at the place up to which the head is measured from the lower edge of the notch.

IV. *Triangular or V-shaped notch, with a still pond*, h_1 measured from the apex of the triangle to the level of still water.

Let a denote the ratio of the *half-breadth* of the notch at any given level to the height above the apex, so that, for example, at the level of still water, the whole breadth of the notch is $2\,a\,h_1$.

$$Q = 8·025\,c \times \frac{8}{15}\,a\,h_1^{\frac{5}{2}} = 4·28\,c\,a\,h_1^{\frac{5}{2}}; \ldots\ldots\ldots (6.)$$

and adopting the values of c already given in Article 448, p. 681, we have,

$$\text{for } a = 1;\ Q = 2·54\,h_1^{\frac{5}{2}}; \ldots\ldots\ldots\ldots\ldots (6\text{ A.})$$

$$\text{for } a = 2;\ Q = 5·08\,h_1^{\frac{5}{2}} \ldots\ldots\ldots\ldots\ldots\ldots (6\text{ B.})$$

In the absence of sufficiently extensive tables of squares and fifth powers, the best method of computing the fifth power of the square root of the head is by the aid of logarithms.

V. *Drowned orifices* are those which are below the level of the water in the space into which the water flows as well as in that from which it flows. In such cases the difference of the levels of still water in those two spaces is the head to be used in computing the flow.

VI. *Drowned rectangular notch.*—Let h_1 and h_2 be the heights of the still water above the lower edge of the notch at the up-stream and down-stream sides of the notch-board respectively; the following formula gives the flow in cubic feet per second:—

$$Q = 5·35\ c\ b \left(h_1 + \frac{h_2}{2}\right) \sqrt{(h_1 - h_2)} \ldots\ldots\ldots\ldots(7.)$$

VII. For *weirs with broad flat crests*, drowned or undrowned, the formulæ are the same as for rectangular notches, except that the co-efficient c is about ·5, as has been stated.

VIII. *Computation of the dimensions of orifices.*—The whole of the preceding formulæ (with the exception of equations 5 and 7) can easily be used in an inverse form, in order to find the dimensions of orifices that are required to discharge given volumes of water per second.

For example, if equation 1 is applicable, we have for the area of the orifice,

$$A = Q \div 8·025\ c \sqrt{h} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(8.)$$

If equation 3 is applicable, the breadth of the orifice is given as follows:—

$$b = Q \div 5·35\ c\ (h_1^{\frac{3}{2}} - h_0^{\frac{3}{2}}). \ldots\ldots\ldots\ldots\ldots(9.)$$

If equation 4 is applicable, the depth of the bottom of the notch below still water is given by the equation,

$$h_1 = \{Q \div 5·35\ c\ b\}^{\frac{2}{3}}; \ldots\ldots\ldots\ldots\ldots\ldots(10.)$$

if equation 6 is applicable,

$$h_1 = \{Q \div 4·28\ c\ a\}^{\frac{2}{5}}. \ldots\ldots\ldots\ldots\ldots\ldots(11.)$$

IX. *Sluices.*—The opening of a sluice generally acts as a rectangular orifice, drowned or undrowned as the case may be; the value of c being as given in Article 448, p. 681.

450. **Computation of the Discharge and Diameters of Pipes.**—The loss of head by a stream of the velocity v in traversing the length

l of a pipe of the uniform diameter d is given by the following formula, deduced from equations 8 and 10 of Article 447, by putting $d \div 4$ for the hydraulic mean depth m:—

$$h = \frac{4fl}{d} \cdot \frac{v^2}{64 \cdot 4} = 0 \cdot 02 \left(1 + \frac{1}{12\, d \text{ (feet)}}\right) \frac{l}{d} \cdot \frac{v^2}{64 \cdot 4} \quad \ldots(1.)$$

From this equation are deduced the solutions of the following problems:—

I. *To compute the discharge of a given pipe;* the data being h, l, and d, all in feet.

For a rough approximation, it is usual to assume an average value for $4f$; say, 0·0258. This gives for the approximate velocity, in feet per second,

$$v = 8 \cdot 025 \sqrt{\frac{h\,d}{\cdot 0258\, l}} = 50 \sqrt{\frac{h\,d}{l}}; \quad \ldots\ldots\ldots(2.)$$

or *a mean proportional between the diameter and the loss of head in 2,500 feet of length;* and for the discharge, in cubic feet per second,

$$Q = \cdot 7854\, v\, d^2 = 39 \sqrt{\frac{h}{l}} \cdot d^{\frac{5}{2}}, \text{ nearly.} \quad \ldots\ldots(2\text{A.})$$

When greater accuracy is required, make

$$4f = 0 \cdot 02 \left(1 + \frac{1}{12\, d \text{ (feet)}}\right); \quad \ldots\ldots\ldots\ldots(3.)$$

and find the velocity in feet per second by the formula

$$v = 8 \cdot 025 \sqrt{\frac{h\,d}{4fl}}; \quad \ldots\ldots\ldots\ldots(4.)$$

and the discharge, in cubic feet per second, by the formula

$$Q = \cdot 7854\, v\, d^2 = 6 \cdot 3 \sqrt{\frac{h}{4fl}} \cdot d^{\frac{5}{2}}. \quad \ldots\ldots(4\text{A.})$$

II. *To find (in feet) the diameter* d *of a pipe, so that it shall deliver* Q *cubic feet of water per second, with a loss of head at the rate of* h *feet in each length of* l *feet.*

Supposing the value of $4f$ known,

$$d = \left(\frac{4 f l Q^2}{39 \cdot 73\, h}\right)^{\frac{1}{5}}. \quad \ldots\ldots\ldots\ldots(5.)$$

But $4f$ depends on the diameter sought. Therefore assume, in the first place, an approximate value for $4f$; say, $4f' = \cdot 0258$. Then compute a first approximation to the diameter by the following formula:—

$$d' = 0 \cdot 23 \left(\frac{l Q^2}{h} \right)^{\frac{1}{5}}. \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6.)$$

From the approximate diameter, by means of equation 3 of this Article, calculate a second approximation, $4f''$, to the value of $4f$. If this agrees with the value first assumed, d' is the true diameter; if not, a corrected diameter is to be found by the following formula:—

$$d = d' \left(\frac{f''}{f'} \right)^{\frac{1}{5}} = d' \cdot \left(\frac{4}{5} + \frac{f''}{5 f'} \right) \text{ nearly.} \quad \ldots\ldots\ldots (7.)$$

In the preceding formulæ the pipe is supposed to be free from all curves and bends so sharp as to produce appreciable resistance. Should such obstructions occur in its course, they may be allowed for in the following manner:—Having first computed the diameter of the pipe as for a straight course, calculate the additional loss of head due to curves by the proper formula (Article 447, p. 678); let h'' denote that additional loss of head; then make a further correction of the diameter of the pipe, by increasing it in the ratio of

$$1 + \frac{h''}{5 h} : 1. \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8.)$$

By a similar process an allowance may be made for the loss of head on first entering the pipe from the reservoir, viz.:—

$(1 + F) v^2 \div 64 \cdot 4$; F being the factor of friction of the mouthpiece.

To the diameter of a pipe, as computed by the formulæ, an addition is commonly made in practice, in order to allow for accidental obstructions, for the incrustation of the interior of the pipe, &c. According to some authorities about one-sixth is to be added to the diameter of the pipe for this purpose; but experience seems to show that in general the incrustation, if any is of equal thickness in pipes of all diameters exposed for equal times to the action of the same water; and therefore that, in a given system of water-pipes, an equal absolute allowance should be made for possible incrustation in pipes of all diameters. In ordinary cases it appears that about *one inch* is sufficient for that purpose. (See p. 803.)

451. **Discharge and Dimensions of Channels.**—The rate of declivity required for the surface of the current in an uniform

conduit or river-channel is found by dividing the loss of head h (which is all actual fall) by the length l of the channel, and is expressed by the following equation, deduced from equation 11 of Article 447, p. 678:—

$$i = \frac{h}{l} = \frac{f}{m} \cdot \frac{v^2}{64 \cdot 4} = \left(\cdot 0071 + \frac{\cdot 00023}{v} \right) \cdot \frac{v^2}{64 \cdot 4\, m}; \quad (1.)$$

m being the "hydraulic mean depth." This equation enables the following problems to be solved:—

I. *To compute the discharge of a given stream*, the data being i, m, and the sectional area A. The first step is to find the velocity, which might be done by means of a quadratic equation; but it is less laborious to find it by successive approximations. For that purpose assume an *approximate value* for the co-efficient of friction, such as

$$f' = \cdot 007565;$$

then the *first approximation* to the velocity is

$$v' = 8 \cdot 025 \sqrt{\frac{i\, m}{\cdot 007565}} = \sqrt{8512\, i\, m} = 92 \cdot 26 \sqrt{i\, m}; \quad (2.)$$

or, *a mean proportional between the hydraulic mean depth and the fall in 8,512 feet.* A *first approximation to the discharge* is

$$Q' = v' A \ldots\ldots\ldots\ldots\ldots (3.)$$

These first approximations are in many cases sufficiently accurate. To obtain second approximations, compute a corrected value of f according to the expression in brackets in equation 1; should it agree nearly or exactly with f', the first assumed value, it is unnecessary to proceed further; should it not so agree, correct the values of the velocity and discharge by multiplying each of them by the factor,

$$\frac{3}{2} - \frac{f}{\cdot 01513} \ldots\ldots\ldots\ldots\ldots (4.)$$

II. *To determine the dimensions of an uniform channel, which shall discharge Q cubic feet of water per second with the declivity* i.—To solve this problem, it is necessary, in the first place, to assume a figure for the intended channel, so that the *proportions* of all its dimensions to each other, and to the hydraulic mean depth m, may be fixed. This will fix also the proportion $A \div m^2$ of the sectional area to the square of the hydraulic mean depth, which will

be known although those areas are still unknown; let it be denoted by n.

[The following are examples of the values of n for different figures of cross-section:—

for a semicircle, $n = 6{\cdot}2832$;
for a half-square, $n = 8$;
for a half-hexagon, $n = 4\sqrt{3} = 6{\cdot}928$;
for a section (proposed by Mr. Neville) bounded below and at the sides by three straight lines, all tangents to one semicircle which has its centre at the water level, the bottom being horizontal, and the sides sloping at any angle θ (see fig. 288);

$$n = 4\left(\operatorname{cosec}\theta + \tan\frac{\theta}{2}\right).$$

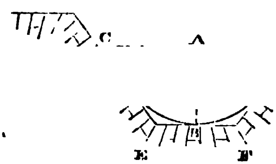


Fig. 288.

In each of the four figures mentioned above, m is one-half of the greatest depth.]

Compute a *first approximation* to the required hydraulic mean depth as follows:—

$$m' = \left(\frac{Q^2}{8{,}512\, n^2\, i}\right)^{\frac{1}{5}};\ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(5.)$$

also a first approximation to the velocity,

$$v' = \frac{Q}{n\, m'^2};\ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(6.)$$

from these data, by means of equation 1 of this article, compute an *approximate declivity* i'. If this agrees exactly or very nearly with the given declivity, i, the first approximation to the hydraulic mean depth is sufficient; if not, a *corrected hydraulic mean depth* is to be found by the following formula:—

$$m = m'\left(\frac{4}{5} + \frac{i'}{5\,i}\right).\ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(7.)$$

From the hydraulic mean depth, all the dimensions of the channel are to be deduced, according to the figure assumed for it.

452. **Elevation Produced by a Weir.**—When a weir or dam is erected across a river, the following formulæ serve to calculate the height h_1, in feet, at which the water in the pond, close behind the weir, will stand above its crest; Q being the discharge in cubic feet per second, and b the breadth of the weir in feet:—

I. *Weir not drowned*, with a flat or slightly rounded crest—

$$h_1 = \left(\frac{Q^2}{7\,b^2}\right)^{\frac{1}{3}},\ \text{nearly.} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1.)$$

II. *Weir drowned.*—Let h_2 be the height of the water in front of the weir above its crest.

$$\textit{First approximation;}\ h'_1 = h_2 + \left(\frac{Q^2}{7\,b^2}\right)^{\frac{1}{3}}. \quad \ldots\ldots (2.)$$

$$\textit{Second approximation;}\ h''_1 = h'_1 - h_2\left(1 - \frac{5}{4} \cdot \frac{h_2}{h'_1 - h_2}\right). \quad (3.)$$

Closer approximations may be obtained by repeating the last calculation.

453. **Backwater** is the effect produced by the elevation of the water-level in the pond close behind the weir, upon the surface of the stream at places still farther up its channel.

For a channel of uniform breadth and declivity, the following is an approximate method of determining the figure which a given elevation of the water close behind a weir will cause the surface of the stream farther up to assume.

Let i denote the rate of inclination of the *bottom* of the stream, which is also the rate of inclination of its surface before being altered by the weir.

Let δ_0 be the natural depth of the stream, before the erection of the weir.

Let δ_1 be the depth as altered, close behind the weir.

Let δ_2 be any other depth in the altered part of the stream.

It is required to find x, the distance from the weir in a direction up the stream at which the altered depth δ_2 will be found.

Denote the ratio in which the depth is altered at any point by

$$\delta \div \delta_0 = r;$$

and let ϕ denote the following function of that ratio:—

$$\phi = \int \frac{dr}{r^3 - 1} = \frac{1}{6}\ \text{hyp. log.} \left\{1 + \frac{3\,r}{(r-1)^2}\right\} \quad (1.)$$
$$+ \frac{1}{\sqrt{3}}\ \text{arc. tan.}\ \frac{2\,r + 1}{\cdots}$$

A convenient approximate formula for computing ϕ is as follows:—

$$\phi \text{ nearly} = \frac{1}{2\,r^2} + \frac{1}{5\,r^5} + \frac{1}{8\,r^8} \cdots\cdots\cdots\cdots (1\text{ A.})$$

Compute the values, ϕ_1 and ϕ_2, of this function, corresponding to the ratios

$$r_1 = \delta_1 \div \delta_0 \text{ and } r_2 = \delta_2 \div \delta_0.$$

Then

$$x = \frac{\delta_1 - \delta_2}{i} + \left(\frac{1}{i} - 264\right) \cdot (\phi_1 - \phi_2)\,\delta_0 \cdots\cdots\cdots (2.)$$

The following table gives some values of ϕ:—

r	ϕ	r	ϕ
1·0	∞	1·8	·166
1·1	·680	1·9	·147
1·2	·480	2·0	·132
1·3	·376	2·2	·107
1·4	·304	2·4	·089
1·5	·255	2·6	·076
1·6	·218	2·8	·065
1·7	·189	3·0	·056

The first term in the right-hand side of the formula 2 is the distance back from the weir at which the depth δ_2 would be found if the surface of the water were level. The second term is the additional distance arising from the declivity of that surface towards the weir. The constant 264 is an approximation to $2 \div f$ f being the co-efficient of friction. For a natural declivity of 1 in 264 the second term vanishes. For a steeper declivity it becomes negative, indicating that the surface of the water rises towards the weir; but although that rise really takes place in such cases, the agreement of its true amount with that given by the formula is somewhat uncertain, inasmuch as the formula involves assumptions which are less exact for steep than for moderate natural declivities. It is best, therefore, in cases of natural declivities steeper than 1 in 264, to compute the extent of backwater simply from the first term of the formula.

454. **Stream of Unequal Sections.**—The preceding rule for determining the figure and extent of backwater is the solution of a particular case of the following general problem:—*Given the form of the bed of a stream, the discharge Q, and the water-level at one cross-section; to find the form assumed by the surface of the water in an* up-stream *direction from that cross-section.*

In this case the loss of head between any two cross-sections is the sum of that expended in overcoming friction, and of that due to change of velocity, when the velocity increases, or the difference of those two quantities when the velocity diminishes, which difference may be positive or negative, and may represent either a loss or a gain of head. In parts of the stream where the difference is negative, the surface slopes the reverse way. In fig. 289, let O Z be the vertical plane of the cross-section at which the water-level is given; let horizontal abscissæ, such as O X = x, be measured *against* the direction of flow, and vertical ordinates to the surface of the stream, such as X B = z, upwards from a horizontal datum plane. Consider any indefinitely short portion of the stream whose length is $d\,x$, hydraulic mean depth m, and area of section A. The fall in that portion of the stream is $d\,z$, and the acceleration = $-d\,v$, because of v being opposite to x. Then,

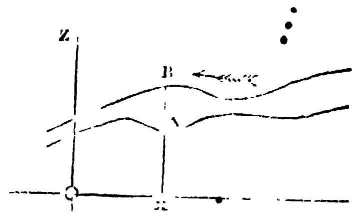


Fig. 289.

$$d\,z = -\frac{v\,d\,v}{32 \cdot 2} + \frac{f\,d\,x}{m} \cdot \frac{v^2}{64 \cdot 1} \text{ ....(1.)}$$

In applying this differential equation to the solution of any particular problem, for v is to be put Q ÷ A, and for A and m are to be put their values in terms of x and z. Thus is obtained a differential equation between x and z, and the constant quantity, Q, which equation, being integrated, gives the relation between x and z, the co-ordinates of the surface of the stream.

455. The **Time of Emptying a Reservoir** is determined by conceiving it to be divided into thin horizontal layers at different heights above the outlet, finding the velocity of discharge for each layer, and thence the time of discharge, and summing or integrating the results.

Let s be the area of any given layer, $d\,h$ its depth, A the effective area of the outlet, h the height of the layer above the outlet; then the velocity of outflow for that layer is $C\sqrt{h}$, C being a multiplier taken from the proper formula in Articles 449, 450, or 451. The time of discharge of the layer is

$$d\,t = \frac{s\,d\,h}{A\,C\sqrt{h}}; \text{ ....(1.)}$$

and if h_1 be the height of the top water, the whole time is,

$$t = \frac{1}{\mathrm{A\,C}} \int_0^{h_1} \frac{s\,d\,h}{\sqrt{h}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.)$$

One of the most convenient ways of expressing this result is to state the ratio which the time of emptying bears to the time of discharging a quantity of water equal to the contents of the reservoir $\left(\text{that is, } \int_0^{h_1} s\,d\,h\right)$, supposing it kept always full. Let that time be called T; its value is $\mathrm{T} = \int_0^{h_1} s\,d\,h \div \mathrm{A\,C}\sqrt{h_1}$, and that of the required ratio is

$$\frac{t}{\mathrm{T}} = \sqrt{h_1} \cdot \int_0^{h_1} \frac{s\,d\,h}{\sqrt{h}} \div \int_0^{h_1} s\,d\,h. \ldots\ldots\ldots\ldots (3.)$$

The following are examples:—

Reservoir with vertical sides ($s =$ constant); $t \div \mathrm{T} = 2$.
Wedge shaped reservoir ($s =$ constant $\times\, h$); $t \div \mathrm{T} = 1\frac{1}{3}$.
Pyramidal reservoir, the base of the pyramid being the surface, the apex at the outlet ($s =$ constant $\times\, h^2$); .... $t \div \mathrm{T} = 1\frac{1}{5}$.

The division of the reservoir into layers may be facilitated by a plan with contour-lines at a series of different levels.

The time required to empty part of a reservoir is found by computing the time required to empty the whole, and subtracting from it the time which would be required to empty the remaining part.

The time required to *equalize the level of the water* in two adjoining basins with vertical sides (such as lock-chambers on canals), when a communication is opened between them under water, is the same with that required to empty a vertical-sided reservoir of a volume equal to the volume of water *transferred* between the chambers, and of a depth equal to their greatest difference of level.

SECTION II.—*Of the Measurement and Estimation of Water.*

456. **Sources of Water in General—Rain-fall, Total and Available.**—The original source of all supplies of water is the rain-fall. The rain-water which escapes evaporation and absorption by vegetables either runs directly from the surface of the ground or from the pores of the surface-soil into *streams*, or it sinks deeper into the ground, flows through the crevices of porous strata, and escapes at their out-crop in *springs*, or collects in such porous strata, from which it is drawn by means of *wells*.

In what manner soever the water is collected, and whether it is

to be used for irrigation, for driving machinery, for feeding a canal, or for the supply of a town, or to be got rid of as in works of mere drainage, the measurement of the rain-fall of the district whence it comes is of primary importance. To complete that measurement two kinds of data are required, —the area of the district, called the *drainage area*, or *catchment-basin*, or *gathering-ground*; and the depth of rain-fall in a given time.

I. *A Drainage Area*, or *Catchment-basin*, is, in almost every case, a district of country enclosed by a *ridge* or *water-shed line* (see Article 58, p. 93), continuous except at the place where the waters of the basin find an outlet. It may be, and generally is, divided by branch ridge-lines into a number of smaller basins, each drained by its own stream into the main stream. In order to measure the area of a catchment-basin a plan of the country is required, which either shows the ridge-lines or gives data for finding their positions by means of detached levels, or of contour-lines. (Article 59, p. 95.)

When a catchment-basin is very extensive it is advisable to measure the several smaller basins of which it consists, as the depths of rain-fall in them may be different; and sometimes, also, for the same reason, to divide those basins into portions at different distances from the mountain-chains, where rain-clouds are chiefly formed.

The exceptional cases, in which the boundary of a drainage area is not a ridge-line on the surface of the country, are those in which the rain-water sinks into a porous stratum until its descent is stopped by an impervious stratum, and in which, consequently, one boundary at least of the drainage area depends on the figure of the impervious stratum, being, in fact, a ridge-line on the upper surface of that stratum, instead of on the ground, and very often marking the upper edge of the outcrop of that stratum. If the porous stratum is partly covered by a second impervious stratum, the nearest ridge-line on the latter stratum to the point where the porous stratum crops out, will be another boundary of the drainage area. In order to determine a drainage area under these circumstances it is necessary to have a geological map and sections of the district.

II. The *Depth of Rain-fall* in a given time varies to a great extent at different seasons, in different years, and in different places. The extreme limits of annual depth of rain-fall in different parts of the world may be held to be respectively nothing and 150 inches. The average annual depth of rain-fall in different parts of Britain ranges from 22 inches to 140 inches, and the least annual depth recorded in Britain is about 15 inches.

The rain-fall in different parts of a given country is, in general, greatest in those districts which lie towards the quarter from which

the prevailing winds blow; and greater in forests than in the open country, in the ratio of $1\frac{1}{2}$ to 1. Upon a given mountain-ridge, however, the reverse is the case, the greatest rain-fall taking place on that side which lies to leeward, as regards the prevailing winds: thus, in Britain, more rain falls in general on the eastern than on the western slope of a range of hills. The cause of this is probably the fact that the condensation of watery vapour in the atmosphere into rain-clouds arises in general from the ascent of moist and warm air up the slopes of mountains into a cold region; the clouds thus formed are drifted by the wind to the leeward side of the mountains, and there fall in rain. To the same cause may be ascribed the fact that the rain-fall is greater in mountainous than in flat districts, and greater at points near high mountain-summits than at points further from them.

The elevation of the locality where the rain-fall is measured does not appear materially to affect the depth, except in so far as elevation is an usual accompaniment of nearness to a mountain-chain.

A vast amount of detailed information has been collected as to the depth of rain-fall in different places at different times; but there does not yet exist any theory from which a probable estimate of the rain-fall in a given district can be deduced independently of direct observation.

The most important data respecting the depth of rain-fall in a given district, for practical purposes, are the following:—

(1.) The least annual rain-fall.
(2.) The mean annual rain-fall.
(3.) The greatest annual rain-fall.
(4.) The distribution of the rain-fall at different seasons, and, especially, the longest continuous drought.
(5.) The greatest flood rain-fall, or continuous fall of rain in a short period.

The order of importance of these data depends on the purpose of the proposed work. If it is one of water-supply, the least annual rain-fall and the longest drought are the most important data; if it is a work of drainage, the greatest annual rain-fall and the greatest flood are the most important.

Experience shows that to obtain those data completely and exactly for a given district requires at least 20 years of daily rain-gauge observations, if not more. But it very seldom happens that so long a series of observations has been made in the precise spots to which the inquiries of the engineer are directed, and in the absence of such records he may proceed as follows:—

(1.) Obtain a copy of the records of the observations made at the

nearest station where the rain-fall has been observed for a long series of years, and from them ascertain the longest drought, and compute the mean annual rain-fall at that station, the greatest and least annual rain-fall, the greatest flood rain-fall, &c. The station in question may be called the "standard station."

(2.) Establish rain-gauges in the district to be examined, at places which may be called the "catchment stations," and have them observed daily by trustworthy persons, taking care to obtain a copy of the records of the observations made at the same time at the standard station; and let that series of simultaneous observations be carried on as long as possible.

(3.) Compute from those simultaneous observations the proportions borne to the rain-fall at the standard station by the rain-fall in the same time at the several catchment stations; multiply the greatest, least, and mean annual depths of rain-fall, the greatest flood, &c., at the standard station by those proportions, and the results will give probable values of the corresponding quantities at the catchment stations.

The positions of the catchment rain-gauge stations must, to a considerable extent, be regulated by the practicability of having them observed once a-day; but they should, as far as practicable, be distributed uniformly over the gathering ground. If it consists of a number of branch basins, there should, if possible, be one or more rain-gauges in each of them. If it is bounded or traversed by high hills, some gauges should be placed on or near their summits, and others at different distances from them.

Each rain-gauge should be placed in an open situation, that it may not be screened by rocks, walls, trees, hedges, or other objects. Its mouth should be as near the level of the ground as is consistent with security. It may be surrounded with an open timber or wire fence to protect it from cattle and sheep.

A rain-gauge for use in the field consists, in general, of a conical funnel, with a vertical cylindrical rim, very accurately formed to a prescribed diameter, such as 10 or 12 inches, and a collecting vessel for the water, usually cylindrical, and smaller in area than the mouth of the funnel. If this vessel is to be used as a measuring vessel also, the ratio of its area to that of the mouth of the funnel is accurately ascertained, and the depth at which the water stands in it is shown by means of a float with a graduated brass stem rising above the mouth of the gauge. Sometimes the rain collected is measured by being poured into a graduated glass measure, which the observer carries in a case. The most accurate method of graduating the measure is by putting known weights of water into it, and marking the height at which they stand (as recommended by Mr. Haskoll in his *Engineering Field-Work*). In performing this

process, the weight of a cubic inch of pure water, at 62° Fahr., may be taken as

252·6 grains.*

The glass measure may either be graduated to cubic inches, which, being divided by the area of the funnel in square inches, will give the depth of rain-fall in inches; or it may be graduated to show at once inches of rain-fall in a funnel of the area employed.

Observations of rain-fall in the field are usually recorded to two decimal places of an inch.

It may be stated as a result of experience, that the proportions of the least, mean, and greatest annual rain-fall at a given spot usually lie between those of the numbers 2, 3, and 4, and those of the numbers 4, 5, and 6.

III. The *Available Rain-fall* of a district is that part of the total rain fall which remains to be stored in reservoirs, or carried away by streams, after deducting the loss through evaporation, through permanent absorption by plants and by the ground, &c.

The proportion borne by the available to the total rain-fall varies very much, being affected by the rapidity of the rain-fall and the compactness or porosity of the soil, the steepness or flatness of the ground, the nature and quantity of the vegetation upon it, the temperature and moisture of the air, the existence of artificial drains, and other circumstances. The following are examples:—

Ground.		Available Rain-fall. ÷ Total Rain-fall.
Steep surfaces of granite, gneiss, and slate,	nearly	1
Moorland and hilly pasture, ...	from	·8 to ·6
Flat cultivated country, ...	from	·5 to ·4
Chalk,..............		0

Deep-seated springs and wells give from ·3 to ·4 of the total rain-fall.

Such data as the above may be used in roughly estimating the probable available rain-fall of a district; but a much more accurate and satisfactory method is to measure the actual discharge of the streams at the same time that the rain-gauge observations are made, and so to find the actual proportion of available to total rain-fall.

457. **Measurement and Estimation of the Flow of Streams.**—There

* This is deduced from the value already given in p. 161 for the weight of a cubic foot of pure water at 62° Fahr., viz., 62·355 lbs. avoirdupois, or 436,495 grains. That value is based on data given in Professor Miller's paper on the "Standard Pound" (*Philosophical Transactions*, 1856); it differs slightly from that formerly fixed by statute but since abolished

are three methods of measuring the discharge of a stream by weir-gauges, by current meters, and by calculation from the dimensions and declivity.

I. The use of *Weir-gauges* is the most accurate method, but it is applicable to small streams only. The weir is constructed across the stream so as to dam up a nearly still pond of water behind it, from which the whole flow of the stream escapes through a notch or other suitable sharp-edged orifice in a vertical plate or board, the elevation of still or nearly still water being observed on a vertical scale in the pond, whose zero-point is on a level with the bottom of the notch, or with the centre of a round or rectangular orifice. For the laws of the discharge of water through vertical orifices, see Article 449, p. 681.

For streams of very variable flow, it appears from the experiments of the late Dr. Jas. Thomson, that the right-angled triangular notch is the best form of orifice (see *Reports of the British Association for* 1861), as it measures large and small quantities with equal precision, and has a sensibly constant coefficient of contraction. Where one such notch is insufficient, he recommends the use of a row of them. The pond may have a flat floor of planks, on a level with the bottom of the triangular notch.

When orifices wholly immersed are used, round or square holes are the best, because their co-efficients of contraction vary less than those of oblong holes (see p. 680). If one round or square hole is insufficient, a horizontal row of them may be used.

A weir-gauge should be placed on a straight part of the channel, because if it is placed on a curved part the rush of water from the outlet may undermine the concave bank of the stream. To prevent the weir itself from being undermined in front, the bottom of the channel below the outlet should be protected by an apron of boards, or a stone pitching, or by carrying the water some distance forward in a wooden shoot or spout, placed so low as not to drown any part of the outlet.

Stream-gauges ought to be observed once a-day at least, and oftener when the flow of the stream is in a state of rapid variation, as it is during the rise and fall of floods.

II. *By Current Meters.*—In large streams the flow can in general be measured only by finding the area of cross-section of the stream, measuring by suitable instruments the velocities of the current at various points in that cross-section, taking the mean of those velocities, and multiplying it by the sectional area. The most convenient instrument for such measurements of velocity is a small light revolving fan, on whose axis there is a screw, which drives a train of wheel-work, carrying indexes that record the number of revolutions made in a given time. The whole apparatus is fixed at

the end of a pole, so that it can be immersed to different depths in different parts of the channel. The relation between the number of revolutions of the fan per minute, and the corresponding velocity of the current, should be determined experimentally by moving the instrument with different known velocities through a piece of still water, and noting the revolutions of the fan in a given time.

A straight and uniform part of the channel should always be chosen for experiments on the velocity of a stream.

When from the want of the proper instrument, or any other cause, the velocity of the current cannot be measured at various points, the velocity of its swiftest part, which is at the middle of the surface of the stream, may be measured by observing the motions of any convenient body floating down, and from that greatest velocity the mean velocity may be computed by the formula given in Article 415, p. 674.

III. *By Calculation from the Declivity.*—For this purpose a portion of the stream must be carefully levelled, cross-sections being taken at intervals; and the discharge is to be calculated by the rules of Division I. of Article 451, p. 687 In order that the result may be accurate, the part of the stream chosen should have, as nearly as possible, an uniform cross-section and declivity, and should be free from obstruction to the current, and, above all, from weeds, which have been known to increase the friction nearly tenfold.

IV. *Estimation of Flow in Different Years.*—The discharge of a stream during a certain period of observation having been ascertained, may be used to compute probable values of its least, mean, and greatest discharge in a series of years, by multiplying it by the proportions borne by the rain-fall in those years as observed at the "standard station" (see Article 456, p. 695) to the rain-fall at the same station during the period when the stream was gauged.

458. **Ordinary Flow and Floods.**—Questions often arise between the promoters of a water-work and the owners and occupiers of land on the banks of a stream as to the distinction between the "ordinary" or "average summer discharge" of a stream and the "flood discharge." The distinction is in general not difficult to draw by an engineer who personally inspects the stream at each time that its flow is gauged; but to provide for the case of such inspection being impracticable, Mr. Leslie has proposed an arbitrary rule for drawing that distinction, which many engineers have adopted. It is as follows:—

Range the discharges as observed daily in their order of magnitude.

Divide the list thus arranged into an upper quarter, a middle half, and a lower quarter.

The discharges in the upper quarter of the list are to be considered as *floods*.

For each of the flood discharges thus distinguished substitute the *average of the middle half of the list*, and take the mean of the whole list, as thus modified, for the *ordinary* or *average discharge, exclusive of flood-waters*.

It appears that the ordinary discharge, as computed by this method in a number of examples of actual streams in hilly districts, ranges from *one-third* to *one-fourth* of the *mean discharge, including floods*; being a result in accordance with those arrived at by engineers who have distinguished floods from ordinary discharges to the best of their judgment, without following rules.

459. **Measurement of Flow in Pipes.**—The *Water Meters*, or instruments for measuring the flow in pipes, now commonly used, may be divided into two classes—piston meters and wheel meters.

A piston meter is a small double-acting water-pressure engine, driven by the flow of water to be measured. That of Messrs. Chadwick and Frost records the *number of strokes* made by the piston, each stroke corresponding to a certain volume of water. That of Mr. Kennedy is so constructed that, by means of a rack on the piston-rod driving pinions, the *distance* traversed by the piston is recorded by a train of wheel-work, with dial-plates and indexes.

An example of a wheel meter is that of Siemens, being a small *reaction turbine* or "Barker's mill," driven by the flow. The revolutions are recorded by a train of wheel-work, with dial-plates and indexes.

Another example of a wheel meter is that of Mr. Gorman, being a small *fan turbine* or *vortex wheel* driven by the flow, and driving the indexes of dial-plates.

The ordinary errors of a good water meter are from $\frac{1}{2}$ to 1 per cent.; in extreme cases of variation of pressure and speed errors may occur of $2\frac{1}{2}$ per cent.

The value of the revolutions of a wheel meter should be ascertained experimentally, by finding the number of revolutions made during the filling of a tank of known capacity. (See Appendix.)

SECTION III.—*Of Store Reservoirs.*

460. **Purposes and Capacity of Store Reservoirs.**—A store reservoir is a place for storing water, by retaining the excess of rain-fall in times of flood, and letting it off by degrees in times of drought. It effects one or more of the following purposes:—

To prevent damage by floods to the country below the reservoir.

To prevent the evil consequences of droughts.

To increase the ordinary or available flow of a stream by adding to it the whole or part of the flood-waters.

To enable water to be diverted from a stream without diminishing the "ordinary" or "average summer flow," as defined in Article 458, p. 698.

To allow mechanical impurities to settle.

The *available capacity* or *storage-room* of a reservoir is the volume contained between the highest and lowest working water-levels, and is less than the *total capacity* by the volume of the space below the lowest working water-level, which is left as a place for the collection of sediment, and which is either kept always full, or only emptied when it is absolutely necessary to do so for purposes of cleansing and repair. It is impossible to lay down an universal rule for the volume of the space so left, or "bottom" as it is called; but in some good examples of artificial reservoirs it occupies about one sixth of the greatest depth of water at the deepest part of the reservoir.

The absolute storage room required in a reservoir is regulated by two circumstances;—the *demand* for water, and the extent to which the *supply* fluctuates.

The demand is usually a certain uniform quantity per day. Experience has shown that about 120 *days' demand* is the least storage-room that has proved sufficient in the climate of Britain; in some cases it has proved insufficient; and even a storage equal to 140 days' demand has been known to fail in a very dry season; and consequently some engineers advise that every store reservoir should if possible contain *six months' demand*.

From data respecting various existing reservoirs and gathering-grounds, given by Mr. Beardmore (*Hydraulic Tables*), it appears that the storage-room varies *from one-third to one-half of the available annual rain-fall*.

The best rule for estimating the available capacity required in a store reservoir would probably be one founded upon taking into account the supply as well as the demand. For example—

180 *days of the excess of the daily demand, above the least daily supply*, as ascertained by gauging and computation in the manner described in the preceding section.

In order that a reservoir of the capacity prescribed by the preceding rule may be efficient, it is essential that the *least available annual rain-fall* of the gathering-ground should be sufficient to supply *a year's demand for water*.

To enable the gathering-ground to supply a demand for water corresponding to the *average available annual rain-fall*, the *greatest*

total deficiency of available rain-fall below such average, whether confined to one year or extending over a series of years, must be ascertained, and an addition equal to such deficiency made to the reservoir room; but it is in general safer, as well as less expensive, to extend the gathering-ground so that the least annual supply may be sufficient for the demand.

The foregoing principles as to capacity have reference to those cases in which the water is to be used to supply a demand for water. When the sole object of the reservoir is to prevent floods in the lower parts of the stream, it ought to be able to contain the ascertained greatest total excess of the available rain-fall during a season of flood above the greatest discharging capacity of the stream consistent with freedom from damage to the country.

461. **Reservoir Sites.**—In choosing the site of a reservoir, the engineer has three things chiefly to consider: the elevation, the configuration of the ground, and the materials, especially those which will form the foundations of the embankment or embankments by which the water is to be retained.

I. The *Elevation* of the site must at once be so high that from the lowest water-level there shall be sufficient fall for the pipes, conduits, or other channels by which the water is to be discharged, and at the same time so low that there shall be a sufficient gathering-ground above the highest water-level.

II. The *Configuration of the Ground* best suited for a reservoir site is that in which a large basin can be enclosed by embanking across a narrow gorge. To enable the engineer to compare such sites with each other, and to calculate their capacities, plans with frequent contour-lines are very useful (Article 59, p. 95), or in the absence of contour-lines, numerous cross-sections of the valleys. The water's edge of the reservoir is itself a contour-line. After the site of a reservoir has been fixed, a plan of it should be prepared with contour-lines numerous and close enough to enable the engineer to compute the capacity of every foot in depth from the lowest to the highest water-level, so that when the reservoir is constructed and in use, the inspection of a vertical scale fixed in it may show how much water there is in store.

Care should be taken to observe whether the basin of a projected reservoir site has, besides its lowest outlet, higher outlets through which the water may escape when the lowest outlet is closed, unless they also are closed by embankments.

The figure of the ground at the site of a proposed reservoir embankment must be determined with care and accuracy, by making not only a longitudinal section along the centre line of the embankment (which section will be a cross-section as regards the valley), but several cross-sections of the site of the embankment, which should

be at right angles to the longitudinal section, unless there is some special reason for placing them otherwise. One of these cross-sections of the embankment site should run along the course of the existing outlet of the reservoir site (usually a stream), and another along the course of the intended outlet (usually a culvert containing one or more pipes).

III. *Material.*—The materials of the site of the intended embankment should be either impervious to water or capable of being easily removed so far as they are pervious, in order to leave a watertight foundation; and their nature is to be ascertained by borings and trial pits, as to which, see Article 187, p. 331, and Article 391, p. 598; and, if necessary, by mines. (Article 392, p. 594.) In many cases it is not sufficient to confine this examination to the site of the embankment; but the bottom and sides of the reservoir-basin must be examined also, in order to ascertain whether they do not contain the outcrop of porous strata, which may conduct away the impounded water. The best material for the foundation of a reservoir embankment is clay, and the next, compact rock free from fissures. Springs rising under the base of the embankment are to be carefully avoided.

The engineer should ascertain where earth is to be found suitable for making the embankment, and especially clay fit for puddle.

462. **Land Awash** means land which lies near the margin of a reservoir, at a height not exceeding three feet above the top water-level, and whose drainage is consequently injured. The promoters of the reservoir are sometimes obliged to purchase such land. Its boundary is of course a contour-line.

463. **Construction of Reservoir Embankments.**—I. *General Figure and Dimensions.*—A reservoir embankment rises at least 3 feet above the top water level, and in some cases 4, 6, or even 10 feet. It has a level top, whose breadth may be in ordinary cases about *one-third* of the greatest height of the embankment; the outer slope, or that furthest from the water, may have an inclination regulated by the stability of the material, such as $1\frac{1}{2}$ to 1, or 2 to 1; the inner slope, or that next the water, is always made flatter, its most common inclination being 3 to 1.

II. The *Setting-out* of the boundaries of the embankment on the ground (see Article 67, p. 113) is to be performed with great accuracy, by the aid of the cross-sections already mentioned in a preceding article. The following method also has been found convenient in suitable situations. On the side of the valley, at one end of the proposed embankment, erect upon props a wooden rail, with its upper edge exactly horizontal, and exactly in the plane of the slope to be set out. At a convenient distance back from the rail as regards the slope, set up a prop supporting a sight having a small eye-hole, also

exactly in the plane of the slope to be set out. A row of pegs ranged from the sight so as to mark points on the ground in a line with the upper edge of the rail will give the foot of the slope.

The same rail (with two different sights) may be used to set out both slopes, if its upper edge coincides with their line of intersection. Let the inner slope be s to 1, the outer s' to 1, the breadth of the top of the embankment b; then the height of that line of intersection above the top of the embankment is,

$$b - (s + s'); \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1.)$$

and its horizontal distance outwards from the centre line of the embankment is,

$$b\ (s - s') \div 2\ (s + s'). \quad \ldots\ldots\ldots\ldots\ldots\ldots (2.)$$

An instrument consisting of a bar with two sights capable of turning about an axis adjusted so as to be perpendicular to the slope to be ranged has been used for the same purpose.

III. *Preparing the Foundation.*—The foundation is to be prepared by stripping off the soil, and excavating and removing all porous materials, such as sand, gravel, and fissured rock, until a compact and water-tight bed is reached.*

IV. The *Culvert* for the outlet-pipes is next to be built in cement or strong hydraulic mortar, resting on a base of hydraulic concrete. Its internal dimensions must be sufficient to admit of the access of workmen beside the pipe or pipes which it is to contain. The principles which should regulate its figure and thickness are those which have been explained in Article 297 A, p. 433. The outer or down-stream end of the culvert is usually open, and often has wing-walls sustaining the thrust of part of the outer slope of the embankment; the inner or up-stream end is usually closed with water-tight masonry, through which the lowest or scouring outlet-pipe passes. In some reservoirs there is a water-tight partition of masonry at an intermediate point in the culvert. The culvert is to be well coated with clay puddle. (Article 206, p. 314.) In the best constructed reservoirs a *tower* stands on the inner end of the culvert, to contain outlet-pipes for drawing water from different levels, with valves, and mechanism for opening and shutting them. Sometimes a cast iron pipe is laid without any culvert.

* The following method was used by Jardine to clear unsound pieces away from the rock foundation of the embankment of Glencorse reservoir, near Edinburgh. A layer of clay puddle was spread and well rammed over the surface of the rock, and was then torn off, when all the fissured fragments came away adhering to the sheet of puddle, leaving a surface of sound rock for the foundation of the embankment.

V. *Making the Embankment.*—The embankment is to be made of clay in thin horizontal layers, as described in Article 199, Division III., p. 311. The central part of the embankment should be a "*puddle wall,*" of a thickness at the base equal to about one-third of its height; it may diminish to about two-thirds or one-half of that thickness at the top. Great care must be taken that the puddle wall makes a perfectly water-tight joint with the ground throughout the whole of its course, and also with the puddle coating of the culvert.*

During the construction of a reservoir embankment care should be taken to provide a temporary outlet for the water of its gathering-ground, sufficient to carry away the greatest flood-discharge. This may be done either by having a pipe sufficient for the purpose traversing the culvert, or by completing a sufficient bye-wash before the embankment is commenced.

VI. *Protection of Slopes and Top.*— The outer slope is usually protected from the weather by being covered with sods of grass. The inner slope is usually *pitched* or faced with dry stone set on edge by hand, about a foot thick, up to about three feet above the top water-level, and as much higher as waves and spray are found to rise. The top of the embankment may be covered with sods like the outer slope; but it is often convenient to make a roadway upon it; in either case it should be dressed so as to have a slight convexity in the middle, like that given to ordinary roads, in order that water may run off it readily.

No trees or shrubs should be allowed to grow on a reservoir embankment, as their roots pierce it and make openings for the penetration of water. For the same reason no stakes should be driven into it.

461. **Appendages of Store Reservoirs.** — I. The *Waste-weir* is an appendage essential to the safety of every reservoir. It is a weir at such a level, and of such a length, as to be capable of discharging from the reservoir the greatest flood-discharge of the streams which supply it, without causing the water-level to rise to a dangerous height. (As to the discharge over a weir, see Article 449, Divisions II., III., VI., and VII., pp. 682 to 684.) The water discharged over the weir is to be received into a channel, open or covered, as the situation may require, and conducted into the natural water-course below the reservoir embankment. The weir is to be built of ashlar or squared hammer-dressed masonry, the bottom of the waste-

* The late Mr. Smith of Deanston rammed and puddled each successive layer of a reservoir embankment by erecting a rail-fence along each side of it, and driving a flock of sheep several times backwards and forwards along it.

Clay puddle may be protected against the burrowing of rats by a mixture of engine ashes, care being taken not to add so much as to make it pervious to water.

channel, directly in front of it, is best protected by a series of rough stone steps, which break the fall of the water. Instead of a waste-weir, a *waste-pit* has in some cases been used; that is to say, a tower rising through or near the embankment to the top water-level; the waste water falls into this tower and is carried away by a culvert from its bottom; but the efficiency and safety of this contrivance are very questionable, for it seldom can have a sufficient extent of overfall at the top.

II. *Waste-sluices* may be opened to assist the waste weir in discharging an excessive supply of water. They may either be under the control of a man in charge of the reservoir, or they may be self-acting. The simplest and best self-acting waste-sluice is that of M. Chaubart, as to which, see *A Manual of the Steam Engine and other Prime Movers*, Article 139, p. 153.

III. *Culvert, Valve-Tower, Bridge, Outlet-Pipes and Valves.*—The culvert and its tower have been mentioned in the preceding article. When the tower is imbedded in the embankment, as it sometimes is, it is called the *valve-pit;* but the best position for it is in the reservoir, just clear of the embankment; and then a light *foot-bridge* is required to give access to it from the top of the embankment.

When the object of a store reservoir is simply to equalize the flow of a stream, in order to protect the lower country from floods, and to obtain an increased ordinary flow available for irrigation and water-power, one outlet-pipe may be sufficient, discharging into the natural water-course below the embankment; but if the water is to be used for the supply of a town, or for any other purpose to which cleanness is essential, there must be at least two outlet-pipes,—the ordinary *discharge-pipe,* which takes the water from a point or points not below the lowest water-level of the reservoir, in order to conduct it to the town or place to be supplied; and the *cleansing-pipe,* which takes the water at or near the lowest point in the reservoir, and discharges it into the natural water-course below the embankment, and is only opened occasionally in order to scour away sediment. The water-course, where such scouring discharge falls into it, must have its bottom protected by a stone pitching. As to the discharge of pipes, see Article 450, p. 684.

The mouthpieces of such pipes should be guarded against the entrance of stones, pieces of wood, or other bodies which might obstruct them or injure the valves, by means of convex gratings. The valves best suited for them are slide valves, as to which, see *A Manual of the Steam Engine and other Prime Movers*, Article 120, p. 124.

IV. The *Bye-wash* is a channel sometimes used to divert past the reservoir the waters of the streams which supply it, when these

are turbid or otherwise impure. Its dimensions are fixed according to the principles of Article 451, p. 685. Its course usually lies near one margin of the reservoir, and is then conveniently situated for receiving the water discharged by the waste-weir.

In some cases, when a reservoir has been made under a stipulation that only the surplus above a certain quantity was to be allowed to flow into it from the streams, the whole of the streams have been conducted past the reservoir in a bye-wash, having weirs or overfalls along its margin, at certain points in its course above the top water-level of the reservoir. The levels of those weirs were so adjusted that when no more than the prescribed quantity flowed down the bye-wash none escaped over the weirs; but when there was any surplus flow in the bye-wash, the water in it rose above the crests of the weirs, and the surplus escaped over them into the reservoir.

V. *Diversion-cuts* are permanent bye-washes for streams that are so impure as to be rejected altogether.

VI. *Feeders* are small channels for diverting either streams or surface drainage *into* the reservoir, and so increasing its gathering-ground. When used to catch surface drainage, they have been found to conduct to the reservoir from *one-quarter* to *one-half* of the rain-fall.

In connection with feeders for diverting streams into the reservoir may be mentioned what may be called a *separating-weir*, the invention of an assistant of Mr. Bateman, and first used in the Manchester water-works. A weir built across the channel of a stream has in front, and parallel to its crest, a small conduit running along its front slope at such a level that when the stream is in flood, and therefore turbid, the cascade from the top of the weir overleaps the conduit, and runs down the front slope into the natural channel, which conveys it to a reservoir for the supply of mills; but when the flow is moderate, the cascade falls into the small conduit, which leads it into a feeder of the store reservoir for the supply of the city.

The horizontal distance x to which a cascade from the crest of a weir will leap in the course of a given fall z below that crest may be thus calculated. The mean velocity with which the cascade shoots from the weir-crest is nearly

$$v = \frac{2}{3} \times 8 \cdot 025 \sqrt{h_1} = 5 \cdot 35 \sqrt{h_1}; \quad \ldots\ldots\ldots\ldots\ldots\ldots (3.)$$

h_1 being the height from the weir-crest to still water in the pond. Then

$$x = \frac{2\, v \sqrt{z}}{8 \cdot 025} = \frac{4}{3} \sqrt{z\, h_1}. \quad \ldots\ldots\ldots\ldots\ldots\ldots (4.)$$

465. **Reservoir Walls.**—Retaining walls are often used at the foot of the slopes of a reservoir embankment; they are of course to be built in strong and durable hydraulic mortar, especially at the foot of the inner slope. As to their stability and construction, see Articles 265 to 271, pp. 401 to 410.

When the gorge to be closed has a bottom of sound rock, a wall of rubble masonry, built in strong hydraulic mortar, may with great advantage, in point of durability, be substituted for an earthen embankment; and this is especially the case when the depth is great, such as 100 feet and upwards. The masonry should be built with great care; and continuous courses should be avoided; for the bed-joints of such courses tend to become channels for the leakage of the water. In designing the profile of the wall, with a view to stability, strength, and economy of material, the following principles are to be followed:—

(1.) The inner face of the wall to be nearly vertical.

(2.) At each horizontal section, the centre of resistance not to deviate from the middle of the thickness, inward when the reservoir is empty, outward when full, to such an extent as to produce appreciable tension at the further face of the wall.

(3.) The intensity of the vertical pressure at the inner face of the wall, when the reservoir is empty, and at the outer face when the reservoir is full, not to exceed a safe limit. That limit may be estimated as nearly equivalent to the weight of a column of masonry —160 feet high for the inner face, and about 125 feet high for the outer face; the reason for making the latter value the smaller being, that owing to the batter of the outer face, the resultant pressure may be considerably greater than the vertical pressure, especially near the base of the wall.

466. **Lake Reservoirs.**—To convert a natural lake into a reservoir it must be provided with a waste-weir, and with one or more outlets at the intended lower water-level, controlled by valves. The outlet or outlets may be made either by building a culvert with pipes in an excavation of sufficient depth, or by tunnelling through one of the ridges that enclose the lake.

Section IV.—*Of Natural and Artificial Water-Channels.*

467. **Surveying and Levelling of Water-Channels.**—The principles which connect the dimensions, figure, declivity, velocity of current, and discharge of a water-channel have already been fully set forth in Articles 444 and 445, pp. 673 to 674, and Articles 451 to 454, pp. 686 to 691. In the present section are to be explained the principles according to which such channels, whether natural or artificial, are constructed, preserved, and improved.

The plans of an existing or intended water-channel require no special remark beyond what has already been stated as to plans in general in the first part of this work, except that in the case of existing streams liable to overflow their banks, they should show the boundaries of lands liable to be flooded, and also of those liable to be laid *awash* (see Article 462, p. 702), and that their utility will be greatly increased by contour-lines. The longitudinal section should be made along the centre line of a proposed channel, and along the line of the most rapid current in an existing channel; and it should show the levels of both banks as well as those of the bottom of the channel, and of the surface of the current in its lowest, ordinary, and flooded conditions. It should be accompanied by numerous cross-sections, especially in the case of existing streams of variable sections; and of those cross-sections a sufficient number should extend completely across the lands flooded and awash, to show the figure of their surface. They should include accurate drawings of the archways, roadways, and approaches of existing bridges, also of existing weirs and other obstructions. The nature of the strata should be ascertained, as for any piece of earthwork, by sinking pits and borings, and, in the case of an existing channel, by probing its bottom also, and the results should be shown on the section and plan.

468. **Regime or Stability of a Water-Channel.**—A water-channel is said to be in a state of *régime* or *stability* when the materials of its bed are able to resist the tendency of the current to sweep them forward. The following table shows, on the authority of Du Buat, the greatest velocities *of the current close to the bed*, consistent with the stability of various materials:—

Soft clay,..	0·25	foot per second.
Fine sand,..	0·50	" "
Coarse sand, and gravel as large as peas,	0·70	" "
Gravel as large as French beans,.........	1·00	" "
Gravel 1 inch in diameter,..................	2·25	feet per second.
Pebbles $1\frac{1}{2}$ inch diameter,..................	3·33	" "
Heavy shingle,....................................	4·00	" "
Soft rock, brick, earthenware,..............	4·50	" "
Rock, various kinds,...........................	6·00 and upwards.	" "

As to the relation between the surface velocity, the mean velocity, and the velocity close to the bed, see Article 445, p. 674.

The condition of the channels of streams which have a rocky bed is generally that of stability. When the bed is stony or gravelly the condition is most frequently that of stability in the ordinary state of the river, and instability in the flooded state.

When the bed is earthy its usual condition is either *just stable and no more*, or *permanently unstable*. The former of these conditions arises from the fact of the stream carrying earthy matter in suspension, so that the bed consists of particles which are just heavy enough to be deposited, and which any slight increase of velocity would sweep away.

The bottom of a river in a permanently unstable condition presents, as Du Buat pointed out, a series of transverse ridges, each with a gentle slope at the up-stream side and a steep slope at the down-stream side. The particles of the bed are rolled by the current up the gentle slope till they come to the crest of the ridge, whence they eventually drop down the steep slope to the bottom of a furrow, where they become covered up, and remain at rest till the gradual removal of the whole ridge leaves them again exposed.

When the banks, as well as the bottom, are unstable, the river-channel undergoes a continual alteration of form and position. If the banks are straight, they soon become curved, for a very slight accidental obstacle is sufficient to divert the main current so that it acts more strongly on one bank than on the other: the former bank is scooped away, and becomes concave, and the earthy matter suspended in the stream is deposited in the less rapid part, so as to make the opposite bank convex. A curved part of a river-channel tends to become continually more and more curved; for the centrifugal force (or rather the tendency of the particles of water to proceed in a straight line) causes the particles of water to accumulate towards the concave bank: the current is consequently more rapid there than towards the convex bank, and it scoops away both the bank and the bottom (unless they are able to resist it), and deposits the material in some slower part of the stream: thus the *line of the strongest current* is always *more* circuitous than the centre line of the channel; and the action of the current tends to make the concave banks more concave, the convex banks more convex, and the whole course of the river more serpentine. This goes on until the current meets some material which it cannot sweep away, or until, by the lengthening of the course of the stream and the consequent flattening of its declivity, its velocity is so much reduced that it can no longer scoop away its banks, and stability is established. In some cases stability is never established; but the river presents a serpentine channel which continually changes its form and position.

One of the chief objects of engineering, in connection with the channels of streams, is to protect their banks against the wearing action of the current, so as in some cases to give them that stability which they want in their natural condition, and in other cases to

give them the additional stability that is required in order to resist an increased velocity of current, produced by improvements in the course and form of the channel.

469. **Protection of River-Banks.**—The most efficient protection to the banks of a stream is a thick growth of water-plants; but as these form a serious impediment to the current, artificial protection must be substituted for them, at least below the average water-level. Above that level a plantation of small willows forms a good defense against the destructive action of floods; but it is not applicable where there is a towing path. The means of artificially protecting river-banks may be thus classed:—I. Fascines. II. Timber sheeting. III. Iron sheeting. IV. Crib-work. V. Stone pitching. VI. Retaining walls. VII. Groins.

I. *Fascines*, already referred to in Article 417, p. 625, are bundles of willow twigs from 9 to 12 inches in diameter: the largest are about 20 feet long, but 12 feet is a more common length: they are tied at every 4 feet, or thereabouts. For the protection of a river-bank *below the low water-level* an "apron" or "beard" is laid, consisting of fascines lying with their length up and down the slope of the bank; the upper ends are fastened down to the bank with stakes about 4 fear feet long; the lower ends are sunk, and held down under water by loading them with stones. To protect the bank *above the low water-level* fascines are laid horizontally in layers, with their butt ends towards the stream, so as to form a series of steps rising at the same rate with the slope of the lower part of the bank, or nearly so (say from 1 to 1 to 3 to 1); each layer is fastened down with three rows of stakes 4 feet long: the heads of the stakes rise 8 inches or thereabouts above the fascines, and are laced or wattled with wicker-work, so as to form a crib for the retention of a layer of gravel.

Fascines usually last 6 years above the low water-level and 10 years below.

II. *Timber Sheeting* may consist either of sheet-piles (already described in Article 404, p. 605) or of guide piles and horizontal planks, described in Article 409, p. 613. The wales of the sheet-piling or the guide-piles of the planking must be tied back to anchoring-plates made of planks buried in a firm stratum of earth at a sufficient distance back from the bank. The holding power of such anchoring-plates depends on the same principles as that of iron anchoring-plates, as to which, see Article 272, p. 410.

III. *Iron Sheeting* has already been described in Article 404, p. 606. It is sometimes used for the faces of quays in navigable rivers, being tied back to anchoring-plates. (Article 272, p. 410.)

IV. As to *Crib-work*, see Article 409, p. 614. When used for a

quay or river-bank its interstices are rammed full of clay and gravel.

V. *Dry Stone Pitching* is used to protect earthen banks, of slopes ranging from that of 1 to 1 to that of 2 to 1, or flatter. It consists of stones roughly squared, and laid by hand in courses. Its thickness is usually from 8 to 12 inches at the top, and increases in going down at the rate of 2 or 3 inches per yard. The foot of the pitching must abut against a foundation sufficient to prevent it from slipping. Such a foundation may be made by sinking a row of oblong baskets, each containing about 2 cubic yards of gravel, or by driving a row of piles with horizontal wales at the inner side of their heads; the strength of the wales is a matter of calculation; they have to resist a maximum pressure = weight of pitching × rise of slope ÷ length of slope, the friction of the pitching on the earth being neglected for the sake of security.

VI. *Retaining Walls* are used chiefly where quays are required, and will be again mentioned further on.

VII. *Groins* are small dykes projecting at right angles to the bank to be protected, and are made either of loose stones, of piles and planks, or of wattled stakes. Each groin protects a portion of the bank of about *five* times its own length, and usually causes the current that sweeps round its point to scoop out an excavation in the bottom of the channel of a breadth equal to about one-quarter of the length of the groin, the material scooped out being deposited in the space between the groins. Groins, besides being an obstruction to the current, are injurious to the regularity of figure and stability of the bottom of the channel, and should only be used as a temporary expedient to protect the banks, until works of a better description can be completed.

470. **Improvement of River-Channels.**—The defects in a river-channel which are to be removed by improvements are usually of the following kinds:—The channel may be too shallow, either generally or in particular places; it may be too narrow, either generally or in particular places; it may even in particular places be too wide, if the breadth is so great as to cause the formation of shoals by enfeebling the current; its declivity may be too flat, either from the existence of obstacles, such as shoals, islands, weirs, ill-constructed bridges, or the like, or from its course being too circuitous; occasionally, but rarely, the declivity may be too steep at particular places, giving rise to a current so rapid as to make it impossible to preserve the stability of the bed; but this defect generally arises from the declivity being too flat elsewhere; it may contain sharp turns, injurious to the stability of the banks; it may be divided into branches, so as to enfeeble the current.

Setting aside for the present *diversions of the course* of a river,

which will be considered in the next article, the works for the improvement of the channel consist mainly of:—I. Excavations to remove islands and shoals, and widen narrow places. II. Regulating dykes, to contract wide shallows. III. Works for stopping useless branches.

Before commencing alterations of any kind in a river-channel careful calculations should be made, according to the principles explained in Section I. of this chapter, of the probable effect of such alterations on the level, declivity, and velocity of the current in different states of the river. The object kept in view should be to obtain a channel either of nearly uniform section, or of a section gradually enlarging from above downwards, with a current that shall be sufficient to discharge flood-waters without overflowing the banks more than can be avoided, and at the same time not so rapid as to make it difficult or impossible to preserve the stability of the channel.

All improvements of river-channels should be begun at the lowest point to be altered, and continued upwards; because every improvement takes effect on the parts of the stream above it.

I. *Excavation* under water, by hand dredging, machine dredging, and blasting, has been described in Article 410, p. 614. When the current is at a low level, it may occasionally be advantageous to excavate parts of the bed by enclosing them with temporary dams as if for foundations (Article 409, p. 611), and laying them dry. Excavation of a muddy, sandy, or gravelly bottom, by the aid of the current, is performed by mooring at the place to be deepened a boat, furnished with a transverse projecting frame covered with boards or canvas; this frame descends to within 3 or 4 inches of the bottom of the channel, and the current, forced through that narrow opening, scoops out the material and sweeps it away. From 30 to 70 cubic yards per day have been excavated in this manner with a single boat.

II. *Regulating Dykes* should be adopted with great caution, and only where the excessive width of the channel is an undoubted cause of shallowness. They should not in any case rise much above the low water-level, lest they contract too much the space for flood-waters. They may be built either of dry stone, with a slope of about 1 to 1, or of wattled piles and gravel. The ordinary rules for the construction of dykes of the latter kind are as follows:—The piles in a double row to be driven into the ground to a depth equal to twice the depth of water; their diameter not less than 1-20th of their length; their distance apart longitudinally to be equal to the depth of water; the distance transversely between the rows of piles to be once and a-half the depth of water. They are to be tied together transversely, and wattled with

willow twigs, and the space between the two rows filled with gravel.

III. The *Stopping of Branches* should be performed at their upper ends. In a gentle current it may be effected by means of an embankment of stones and gravel, advancing simultaneously from the two banks until it is closed in the centre; in a more rapid stream a dyke of wattled piles and gravel, made as already described, may be used; should the current be too strong for either of these plans, a raft, boat, or caisson (Article 409, Division III., p. 613), or a crib-work dam (Article 409, Division IV., p. 614), loaded with stones, is to be moored across the stream and sunk. The branch channel having had its current stopped will silt up of itself.

471. **Diversions of River-Channels** are usually adopted for the purpose of rendering the course less circuitous. In designing them regard should be had to the principles already explained in Section I. of this chapter, and in the preceding articles of this section, and care should be taken not to make the course *too direct*, lest the current be rendered too rapid for the stability of the bed. A slightly curved channel is always better than a straight channel; because in the former the main current takes a definite course, being always nearest the concave bank; whereas in a straight channel its course is liable to keep continually changing.

The form of cross-section with a horizontal base and sloping sides which gives the least friction with a given area has already been described in Article 451, p. 688, and it may be adopted if the stream is to act solely as a conduit for the conveyance of water; but should it be navigable, a figure must be adopted suited to the convenience of the navigation. This will be further considered in Chapter III. of this part.

472. A **Weir** is an embankment or dam, usually of stone, sometimes of timber, constructed across the channel of a stream. As to its effect on the water-level, see Articles 452 and 453, pp. 689 and 690.

When erected for purposes of water power or water-supply, the object of a weir is partly to make a small store reservoir, but principally to prolong a high top water-level from its natural situation at a place some distance up the stream, to a place where water is to be diverted from the stream to drive machinery, or for some other purpose. When erected for purposes of navigation, the object of a weir is to produce a long reach or pond of deep and comparatively still water, in a place where the river is naturally shallow and rapid.

In planning a weir three things are to be considered: its line and position, its form of cross-section, and its construction.

I. *Line and Position of a Weir.*—It is best to avoid sharply curved parts of a river-channel in choosing the site of a weir, lest the rapid current which rushes down its face in times of flood should undermine the concave bank. For the protection of the banks in any case, it is advisable so to form the weir that the cascade from the lateral parts of the crest shall be directed from the banks, and towards the centre of the channel. This may be effected either by making the weir slightly curved in plan, with the concavity at the down-stream side, or by making it like a V in plan, with the angle pointing up stream. Another mode of protecting the banks is to make the crest of the weir slightly higher at the ends than in the middle, so that the lateral parts of the cascade may be too feeble to do damage.

In order to diminish the height and extent of backwater during floods, the crest of the weir is often made considerably longer than the breadth of the channel; this is effected either by making it cross the channel obliquely, or by using the V-shape already described, the latter method being the best for the stability of the banks. The practical advantage of such increased length is doubtful.

II. *Form of Cross-section.*—The back or up-stream side of a weir is usually steep, ranging from vertical to a slope of about 1 to 1; the top is either level or slightly convex, and not less than about 2 or 3 feet broad. In designing the front or down-stream slope of a weir, the principal object is to prevent the cascade that rushes over it from undermining its base. The commonest method is to use a long flat slope of 3 to 1, 4 to 1, or 5 to 1, in order that the speed of the current may be diminished by friction, and that it may strike the bottom of the channel very obliquely. A further protection is given to the river-bed by continuing the front slope a short distance below the bottom of the channel, and then curving it slightly upwards. Another method is to make the front of the weir present a steep or nearly vertical face, over which the water falls on a nearly level apron or pitching of timber or stone. Probably the best method would be to form the front of the weir into a series of steps, presenting steep faces and flat platforms alternately, the general inclination being about 3 to 1; thus a great fall might be broken up into a series of small falls, each incapable of damaging the platform which receives it.

III. *Construction.*—In order that the water of the pond may not force its way under the base of a weir, or round its "roots" (as the ends which join the banks of the stream are called), its foundation should be examined, chosen, and formed with precautions similar to those used in the case of a reservoir embankment, as to which, see Articles 461 and 463, pp. 701 to 704.

To make a weir of *timber*, or of timber, stones, and clay combined, any of the methods may be employed which have been described under the head of "Dams," in Article 409, Divisions II., III., and IV., with the addition that the back, crest, and front of the dam are to be covered with planking laid parallel to the current, to form an overfall for the water; and that the bottom of the channel at the foot of the weir is to be protected either by a platform of planks resting on a timber grating or on piles, or by a stone pitching.

A weir of *fascines* may be built of horizontal layers of fascines, staked down with mixed clay and gravel packed between them, in the manner described under the head of the protection of river-banks. Article 469, p. 710, the crest, front, and foot of the dam being protected with an apron of fascines, like that described in the same article.

A *dry stone* weir is formed like the stone embankments mentioned in Article 412, p. 617, with a steep slope at the bank and a long gentle slope in front, pitched or faced with roughly squared stones set in courses, as in the pitching of a river-bank, Article 469, p. 711. Sometimes a skeleton crib of timber, consisting of piles and longitudinal and transverse horizontal wales is constructed in order to keep the stones of the pitching in their places As to the pressure against the longitudinal wales, see the article just quoted.

A weir of *solid masonry* may be founded, like other structures under water, on the natural ground, on a bed of concrete, on a timber platform, or on piles, according to circumstances. (See Part II., Chapter VI., Section II, p. 601) When it has a timber foundation, a row of sheet-piles at the base of the up-stream side will in general be necessary to prevent the passage of water under it; and in the grating of the platform, pieces of timber running continuously through the weir in the direction of the stream should be avoided, lest they should conduct water along their sides. The masonry should be built in cement, or in quickly-setting hydraulic mortar; the heart of the weir may be of coursed rubble, or of concrete laid in layers; but the facing should be of good block-in-course, or of hammer-dressed ashlar, and the crest should form a coping of large stones, all headers, dowelled to each other.

One of the most effectual ways of preventing filtration round the "roots" of a weir is to carry them a considerable distance into the bank; but in the case of a weir of masonry the ends often abut upon a pair of side-walls, running along the banks of the stream, and having counterforts behind them to interrupt filtration.

IV. *Appendages of a Weir—Sluices and Floodgates—Salmon-*

stair.—When a weir is built across a navigable river, it requires a lock for the passage of vessels, which will be again mentioned further on. It may have one or more outlets with valves, like those of a reservoir embankment (Article 461, p. 705), according to the purpose for which it is intended.

It is almost always necessary to provide a weir with waste-sluices or floodgates, to be opened when the river is high, in order to prevent too great a rise of backwater. A sluice is a sliding valve of timber or iron, moving in guides, which are in general vertical, set in a rectangular passage of timber or masonry, and opened and shut by means of a screw, or of a rack and pinion. It is advisable not to make any sluice wider than about 4 or 5 feet. Should a greater width of opening be required, the passage through the weir is to be divided by walls or piers into a sufficient number of parallel passages, each furnished with a sluice. As to the discharge through a sluice, see Articles 448, 449, p. 681.

Another mode of opening and closing floodgates in a weir is by means of *needles*, as they are called. A rectangular channel through the weir is crossed at the bottom by a fixed timber sill, and near the top by a moveable timber sill, resting in two notches. The strength of the sills is a matter of calculation: they have to withstand the pressure of the water on a flat surface closing the passage That surface is made up of the "needles," which are a set of square bars of wood strong enough to withstand the pressure, which are ranged close together side by side in a vertical position at the up-stream side of the sills. Each needle has a cylindrical handle at its upper end, to hold it by in removing and replacing it.

As to *self-acting waste-sluices*, see *A Manual of the Steam Engine and other Prime Movers*, Article 139, p. 153.

A weir across a river frequented by salmon requires a passage or channel to enable those fish to ascend its front slope. Mr. Smith of Deanston introduced the practice of making that channel of a zig zag form, so as to reduce its rate of declivity and bring the speed of the current in it within moderate limits.

A *moveable weir* consists in general of a water-tight planked timber gate, placed in a rectangular passage of masonry or timber, and capable of turning upon a horizontal hinge at the floor of the passage, so as to be either laid flat when the channel is to be left clear, or set at any required angle of elevation, sloping against the declivity of the stream, with oblique struts to prop it at the down-stream side. In one ingenious modification of this weir the duty of the struts is performed by a second and smaller gate, also turning on a horizontal hinge at the floor of the passage, but so as to slope *with* the stream. When the passage is clear, both gates lie flat in a horizontal recess in the floor of the passage, the smaller gate

undermost and the upper surface of the larger gate flush with the floor. When the weir is to be raised, water is admitted through a valve and culvert from the up-stream side of the weir passage into the recess below the gates; its pressure lifts them both until they form a weir of a triangular section, the larger gate making the up-stream slope and the overfall, and the smaller making the down-stream slope, and acting at the same time as a strut to prop the larger gate. When the weir is to be lowered, the mass of water contained below the gates is allowed to escape by opening a valve in a culvert which leads to the down stream side of the weir; and both gates then fall flat into the recess of the floor.*

473. **River Bridges.**—The construction of the foundations on land and in water, and of the superstructures, of bridges of various materials having been explained in Part II. of this work, and their adaptation to roads and railways in the preceding chapter, it is now only necessary to state those principles which are specially applicable to bridges over rivers.

In choosing the site of a bridge which is to have piers in the river, sharply curved parts of the channel should be avoided, lest the increased rapidity of the current caused by the narrowing of the water-way should undermine the concave bank.

The current should be crossed at right angles, or as nearly so as practicable. The abutments should not contract the water-way.

The piers, if any, should stand with their length exactly in the direction of the current, they should have pointed or cylindrical cutwaters at both ends, to diminish the obstruction to the current which they produce; and they should be no thicker than is necessary for the safety of the bridge. (As to stone piers in particular, see Article 293, p. 428.)

The springing of the arches should be above the highest ordinary water-level, and as much higher as the convenience of the navigation may require; and care should be taken that sufficient water-way is provided for the greatest floods. The crown of the lowest arches should be at least three feet above the flood level, that they may allow floating bodies to pass through.

It may here be observed that the figure of arch which gives the greatest water-way for a given rise and span is the "hydrostatic arch." (See Article 283, p. 419.)

* In order to do away as far as possible with the obstruction occasioned by weirs, it has been proposed by Hugh Mackenzie, Esq. of Ardross, that in those cases in which the fall of the stream is sufficiently rapid, and the country in other respects suitable, the diversion of water from a stream for the purpose of obtaining power should be effected by making a tunnel with suitably formed grated apertures in its roof, under the bed of the stream, at a point where its water-level has sufficient elevation, and so conducting the water into a mill-lead of sufficiently large size and moderate declivity.

Should it appear, upon an examination of the land subject to inundation at and near the site of the intended bridge, that such land acts not merely as a reservoir for flood-waters, but as a wide temporary channel for their discharge, that land should be crossed by a viaduct, and not by embanked approaches.

In designing a bridge for carrying an ordinary road over a river, it is usual, in order to obtain the greatest headroom possible consistent with economy in forming the approaches, to give the roadway an ascent from the ends of the approaches to the middle of the bridge, at a rate not exceeding the ruling gradient of the road; and to suit the arches, when there are more than one, to the form of the roadway, the centre arch is made the largest, and the others gradually diminish in size towards the ends of the bridge. They should, at the same time, be so proportioned as to exert as nearly as possible equal horizontal thrust.

Swing bridges for navigable rivers will be again mentioned further on.

Ice breakers are required for the protection of the piers of bridges across rivers which bring down large masses of ice.

A *stone ice-breaker* usually forms part of the up-stream cut water of the pier to which it belongs, presenting to the current a ridge sloping at about 45°, up which the flat sheets of ice slide, and break asunder by their own weight. Examples of such ice-breakers are shown in the view of the Victoria Bridge, fig. 249, p. 533.

A *timber ice-breaker* stands usually separate from the pier which it protects, at a short distance up-stream. The sloping ridge is formed by a beam of 12 or 14 inches square, covered with sheet iron. Its base consists of piles, ranged in the form of a long sharp triangle with the point up-stream, connected with the ridge by a strong framework of uprights and diagonals, which are protected against the ice by projecting horizontal wale.

(On the subject of river bridges, see Telford's and Smeaton's *Reports*, and the work *On Bridges* by Mr. Hosking and others).

474. **Artificial Water-Channels — Conduits.** — In laying out and designing artificial water-channels it is advisable, if possible, so to fix the declivity with reference to the length, that the velocity shall not be less than about one foot per second (lest the conduit silt up), nor greater than about four feet per second (lest the current should sweep stones along, and injure the bed).

As to the larger-sized artificial water-channels, and as to those of all sizes which are merely to be used as open drains, when they are wholly in cutting, it is unnecessary to add anything to what has already been stated respecting river-channels, and especially respecting their diversions, Article 471, p. 713. Artificial earthen

channels in embankment will be considered under the head of canals.

When a channel is to convey water for the supply of a town, it is usual, with a view to the clearness and purity of the water, as well as to the preservation of the channel, to line it throughout with brick or stone built in cement; and in most cases it is necessary to cover it also, especially if it traverses districts where the air is smoky and otherwise impure. When brick or porous stone is used, the water-way may be lined throughout with a coating of cement, calcareous or asphaltic.

The water-way of a *stone or brick conduit* should be made of one of those forms which give the greatest hydraulic mean depth for a figure of given class and a given area; that is to say, the semicircle, the half-square, or the half-hexagon, already referred to in Article 451, p. 688. To preserve a constant definite flow it may have a series of waste-weirs along its sides, placed in positions where there are convenient channels at hand for discharging the waste water. Should it be necessary to carry it along an embankment, that embankment should be formed in thin layers, each well rammed, and should if possible contain a large mixture of stones with the earth; the breadth at the top should be from 4 to 6 feet at each side of the conduit, so that the total breadth at the brink of the conduit will be = breadth of water-way + from 8 to 12 feet, and the masonry of the conduit should be imbedded in puddle or in hydraulic concrete.

The best form for a *covered conduit* to convey a constant flow, as for the supply of a town, is cylindrical. To guard it against frost it should be completely covered with earth to the depth, in Britain, of about 3 feet, the bank being faced with sods. When it forms a tunnel, or is placed in deep cutting and covered with earth, its strength is regulated by the principles of Article 297 A, p. 433.

One of the largest cylindrical conduits yet executed is that of the Loch Katrine Water-Works, 8 feet in diameter.

A covered conduit should be provided, like a tunnel, with grated ventilating shafts, which will also serve to admit men for the purpose of repairing it.

When the flow varies very much, as in sewers, an egg shaped section with the small end down is preferred.

A recent invention in conduits is that of Mr. Richardson, in which a cylinder of sheet iron is lined with brickwork in cement. It is suitable for making large conduits possessing great strength and stability with a moderate quantity of materials.

475. **Junctions of Water-Channels.**—In all cases in which a pair of water-channels join together into one, their centre lines, if

possible, should be a pair of curves, or a curve and a straight line touching each other at the junction; or should an angle at the junction be unavoidable, that angle ought to be as acute as possible. This principle applies also to the *divergence* of a branch from a main channel, and to pipes as well as to free channels.

476. **Aqueduct Bridges** differ from viaducts only in supporting a water-conduit instead of a road or a railway, and the mechanical principles of their construction involve nothing that has not been already explained in the Second Part of this treatise.

The water conduit or trough is usually of the same material with the rest of the structure. For example, in a stone aqueduct the conduit is of masonry, imbedded in a mass either of puddle or of concrete, resting on the arch and contained between the external spandril walls.

In some recent examples of wrought iron aqueducts introduced by Mr. Simpson, the water-channel has been made self-supporting by constructing it as a plate iron tubular girder of oval section. In this case the interior of the tube should be smooth, that it may offer no impediment to the current. All T-iron stiffening-ribs, &c., should project outside only.

Pipe-aqueducts will be mentioned further on.

477. **Water-Pipes.**—The diameters of water-pipes are fixed with reference to the virtual declivity and the intended greatest discharge, according to the rules explained in Article 450, p. 681. The materials principally used in making pipes for the conveyance of large quantities of water are earthenware and iron.

I. *Earthenware Pipes* are of various qualities as to texture, from a porous material like that of red bricks, to a hard and compact material, which is glazed to make it water tight. They are made of various diameters, from 2 inches to nearly 3 feet, and in lengths of from 1 foot to 3 feet. The harder kinds have considerable tenacity, and are capable of bearing the dead pressure of a high column of water; but they are so easily broken by sharp blows and sudden shocks that it is not advisable to expose them to high pressures in situations where their bursting might cause damage or inconvenience. Hence their chief use is as *small covered conduits* for purposes of drainage. Their joints are most commonly of the spigot and faucet form, being made tight, if necessary, with cement, or with a bituminous mastic. (Article 234, p. 376.) Another form, very useful to facilitate laying and lifting is the *thimble-joint*. The lengths of pipe are plain hollow cylinders, and the thimble is a ring embracing and loosely fitting the adjoining ends of a pair of lengths. Sometimes the thimble is in two semicircular halves; and sometimes each pipe has on one end a half-faucet, which is laid downwards; the end of the adjoining pipe rests in the half-faucet,

and the joint is completed by a half-thimble above. Curved and acute-angled junction-pieces are made;* so also are right-angled junction-pieces; but these last should never be used.

II. *Cast Iron Pipes* should be made of a soft and tough quality of cast iron. (See Article 353, p. 499.) Great attention should be paid to moulding them correctly, so that the thickness may be exactly uniform all round. Each pipe should be tested for air-bubbles and flaws by ringing it with a hammer, and for strength by exposing it to double the intended greatest working pressure.

Cast iron water-pipes are made of various diameters of bores, from 2 inches to 4 feet.

They are usually moulded and cast horizontally, the sand core being supported by a strong horizontal bar with projecting teeth; but advantages in point of accuracy and soundness are possessed by the process of casting them vertically, the faucet being turned downwards, and the plain end upwards.* The pipe is cast with an additional length at the upper end, which acts as a *head* (Article 354, p. 503), compressing the mass below, and receiving the air-bubbles; this *head* is afterwards cut off. (See p. 795.)

The rule for computing the thickness of a pipe to resist a given working pressure (the factor of safety being *six*) has already been given in Article 150, equation 2, p. 228, the pressure and the tenacity of the iron being expressed in lbs. per square inch; but as it is more convenient to express those quantities in *feet of water*, the following rule is given:—

$$\frac{\text{thickness}}{\text{diameter}} = \frac{\text{greatest working pressure in feet of water}}{12{,}000}. \quad (1.)$$

There are limitations, however, arising from difficulties in casting, and from the fact that the most severe strain on a pipe is often produced by shocks from without, which cause the thickness of cast iron pipes to be often made considerably greater than that given by the above rule. The following empirical rule expresses very accurately the *limit to the thinness of cast iron pipes*, in ordinary practice:—

The thickness of a cast iron pipe is never to be less than a mean proportional between its internal diameter and one-forty-eighth of an inch.

It is very seldom, indeed, that a less thickness than 3-8ths of an inch is used for any pipe, how small soever.

Cast iron pipes are made of various lengths; but the most common length is 9 feet, exclusive of the faucet or socket on

* Introduced by Mr. D. Y. Stewart.

one end of each length, for receiving the plain end of the next length. The faucet adds from one-twentieth to one-tenth to the weight of the pipe. The joints are sometimes run up with melted lead, sometimes turned so that the plain end and the faucet fit exactly, and made water-tight with red lead paint. The latter is the easier and quicker process; but the former admits of a greater amount of yielding to expansion and contraction, and to the unequal settlement of the ground, which is an advantage in point of safety.

III. The best *preservative* for cast iron pipes against corrosion is a coating of pitch, applied both inside and out, by a process which makes it penetrate the pores of the iron to a certain extent, and adhere very firmly. This coating appears to diminish sensibly the friction of the water. (See pp. 795 and 809.)

IV. In estimating the greatest working pressure which a water-pipe should be capable of resisting, the *hydrostatic pressure* due to the whole depth below top water of the reservoir whence the supply enters the pipe, and not the mere *hydraulic pressure* when the water is in motion (Article 446, p. 675), should be taken into account, in order to provide for the contingency of the flow of the water being checked by an obstruction in the pipe.

V. The *loss-of head* during the most rapid discharge should be computed for a series of points in the course of an intended pipe by the principles explained in the First Section of this chapter, so as to determine the *line of virtual declivity*, which will commence at a point vertically above the mouthpiece of the pipe, and at a depth below the top-water of the reservoir equal to the loss of head due to the velocity of flow in the pipe and the friction of the mouthpiece. The object of determining that line is to insure that in laying out the levels of the pipe *no part of it shall be made to rise above the line of virtual declivity.* The reason for this rule is, that at all points in a pipe which are above that line, the pressure, when the water is flowing, becomes less than that of the atmosphere (a fact commonly described by saying that there is a "partial vacuum," see Article 443, p. 673); in consequence of which the air, which all water contains in a diffused state, escapes from the water in bubbles, and eventually accumulates in the highest part of the pipe so as to obstruct the flow of the water.

A pipe thus rising above the line of virtual declivity is called a *siphon*, and is incapable of continuously conveying water unless the air be from time to time exhausted from the summit of the pipe.

Air collects to a certain extent at the *summits* of an undulating pipe even when they are below the line of virtual declivity; but as it exerts a pressure greater than that of the atmosphere, it is easily expelled. A small cylindrical receiver, called an *air-lock*, is placed above the pipe at each such summit, to collect the air,

which is from time to time discharged through a valve. That valve may either be opened by hand occasionally, or it may be loaded with a weight equivalent to the hydraulic pressure, and made self-acting. (See p. 804.)

VI. At the *lowest points* in an undulating line of water-pipe sediment collects, and is to be discharged from time to time through a *cleansing* or *scouring* cock or valve.

VII. As to *slide-valves, double-beat-valves*, and other valves and cocks used in connection with water-pipes, see *A Manual of Prime Movers*, Article 116, p. 120, and Articles 119 to 123, pp. 123 to 126.

VIII. *Sheet Iron Water-Pipes* lined with pitch have lately been used in France. (See p. 805.)

478. **Pipe-Track—Pipe-Aqueducts.**—Care should be taken to bed water-pipes on a firm foundation, and to cover them to a sufficient depth to prevent the action of frost; that is, in Britain, about 2 or 3 feet.

When a water-pipe crosses a valley, or a river-channel, or a line of communication, it may sometimes be advisable to carry it above ground by means of an aqueduct. This may be a bridge of any convenient construction, or it may consist simply of the pipe itself lying on a series of piers, and cased outside with wood, or other non-conducting material, for protection against heat and cold. For a pipe-aqueduct of wide span, the pipe itself may be made to form a catenarian arch.*

The total thrust at the springing of the arch under an uniform load is to be computed in the usual way, being,

load per foot of span × radius of curvature at crown in feet × secant of inclination at springing;

from which has to be deducted the thrust borne by the water, viz.,

pressure of water × sectional area of pipe;

and the *remainder* only of the thrust has to be borne by the iron of the pipe. In fact, the *arch of water* bears a part of the load.

If the arched pipes be made to carry a roadway, the whole of the stress produced by a partial or travelling load will fall on them; and their strength is to be computed by the formulæ of Article 180, Problems IV. and V., pp. 303 to 308, as explained in treating of cast iron arched ribs, Article 374, Case I., p. 539.

The *wooden lining* referred to as a protection against frost

* Of this there is an example on the Washington Water-Works, designed by General Meigs of the United States' Engineers. The arch is of 200 feet span, and consists of two parallel cast iron pipes of 4 feet diameter.

consists of oaken staves about 3 inches thick, packed in a cylindrical form round the interior of each pipe. It is likely to prove more lasting than an outside casing, because it is constantly wet, instead of being alternately wet and dry.

Section V.—*Of Systems of Drainage.*

479. **General Principles as to Land Drainage.**—The engineer who examines a district with a view to the improvement of its drainage requires the information respecting the features, extent, and levels of the district, its rain-fall, and the course, dimensions, levels, and discharge of its streams, which have already been specified in Articles 456, 457, and 458, pp. 692 to 699, and in Article 467, p. 707. In some cases it is necessary to attend to the question, whether the water to be carried off by the system of drainage comes merely from the apparent gathering-ground bounded by the ridges that surround the district, or whether some of it is brought to the district through porous strata, which have their gathering-ground wholly or partly beyond such ridges.

In order that a district may be in a perfect state as to drainage, the water-level in the branch drains, which directly receive the discharge of the field drains, should be at least about 3 feet below the level of the ground at all times. When it rises above that level the ground becomes *awash* or *flooded*, according as the water-level is below or above its surface.

Each water-channel must have sufficient area and declivity, when at its fullest flow, to discharge all the water that it receives as fast as such water flows in, without its water-level rising so high as to obstruct the flow of the branches it receives, or to lay land awash.

Should it be impossible absolutely to fulfil these conditions, means are to be taken to make the deviation from them as small in extent and as short in duration as possible.

480. **Questions as to Improvement of Drainage.**— Should the drainage of a district be found defective, the engineer will in general have to consider questions of the following kind, as to the causes of such defective condition, and the means of improving it:—

I. Whether, and to what extent, it is practicable to diminish or prevent floods by the construction of store reservoirs.

II. Whether the channels of the streams contain *removable obstructions* such as shelves of rock or other shallows, narrow places, islands, ill-designed weirs and bridges, &c., and how such obstructions are to be removed. This may involve questions as to rebuilding weirs and bridges according to improved designs.

III. Whether the channels are defective and liable to be

obstructed through the instability of their beds, and how such instability is to be prevented.

IV. In the case of a smaller stream having too little declivity, which falls into a larger stream, whether that declivity can be increased by diverting the course of the smaller stream so as to remove its outfall to a lower part of the larger stream.

V. Whether the course of a stream, being too circuitous, can be improved by a diversion; and whether, in the event of improvements being required in the channel of a stream it is best to execute them in the existing channel, or to make a new channel, independently of the question of circuitousness.

All the preceding questions relate to matters which have already been treated of in Sections III. and IV. of this chapter, but the following involve subjects which will be treated of in the ensuing articles:—

VI. Whether the branch drains are of sufficient discharging capacity.

VII. To what extent the water-channels are capable of acting as temporary reservoirs for moderating the rapidity with which flood-waters descend from them into lower and larger channels.

VIII. To what extent the lands adjoining a river which are liable to inundation act in the capacity of a reservoir, and what will be the effect upon the part of the river below them of preventing or diminishing such action.

IX. Whether the drainage can be sufficiently improved by improvements on the water-channels alone, or whether, on the other hand, it is advisable to use embankments for the confinement of floods within certain limits.

481. **Discharging Capacity of Branch Drains.**—If the rain-fall found its way at once from the surface of the ground to the drains, each of these would require to have dimensions and declivity sufficient to discharge the most rapid fall of rain known to take place for any time how short soever. The following data as to the most rapid rain-fall in Britain are given on the authority of Mr. Phillips; they illustrate how the greatest *rate* of rain-fall diminishes according as the period for which it is reckoned is increased:—

Period.	Total depth of Rain-fall. Inches.	Rate of Rain-fall. Inches per Hour.
One hour, ...	1	1·0
Four hours, ...	2	0·5
Twenty-four hours, ...	5	0·2 nearly.

The soil, however, acts as a sort of reservoir to an extent depending on its texture; it keeps from the drains altogether a portion of

the rain-fall, which passes off by evaporation, or is absorbed by plants, as stated in Article 456, p. 692; and it discharges the remainder into the drains more or less gradually. The branch drains in country drainage should be made capable of discharging at an uniform rate the greatest *available* rain-fall known to take place in a period whose length is greater according as the soil is more retentive. It is probable that in most cases of cultivated land *twenty-four hours* will be found a sufficiently short period: that is, each drain which directly receives water from the fields should be capable of discharging, in twenty-four hours, the greatest available rain-fall of twenty-four hours; for steep and rocky ground the period must be shortened, in some cases, it is probable, to four hours; but the best method in each case is to ascertain the period by an experimental comparison of the rain-fall with the discharge of drains.

482. **Action of Channels and Flooded Lands as Reservoirs.**—The volume of the space contained between the ordinary water surface of a given portion of a stream and the flood-water surface, whether such space be wholly contained between the banks of that portion of the stream, or partly between such banks and partly over adjoining lands liable to inundation, constitutes a reservoir for retaining *the excess of the total supply of water during a period of flood rain-fall from the district drained by that portion of the stream, above the greatest quantity that the stream is capable of discharging in the same period*, until the flood rain-fall is over, when that excess flows away by degrees. The existence of that reservoir-room thus renders sufficient a water-channel of less discharging capacity than would otherwise be necessary; and if such reservoir-room is diminished, either by improving the channel so as to lower the flood-water surface, or by contracting the space by means of embankments, care should be taken that the discharging capacity of the channel *below* the district in question is increased to a corresponding extent, otherwise the effect of diminishing the extent of floods in that district may be to increase it in some district further down the river. This is one of the reasons for the rule already stated in Article 470, p. 712, that works of river improvement should proceed from below upwards.

483. **River Embankments.**—When the land adjoining a stream cannot be sufficiently guarded from inundation by improvements in the channel, embankments may be erected. In determining the course and site of such embankments regard must be had to the principle stated in the last article—of leaving sufficient reservoir-room between them for flood-water. In some cases there may be sufficient room even when the embankments are erected close to the natural banks of the channel; but in general it is advisable to

leave a wider space; and when the river follows a serpentine course sufficient reservoir-room may in many cases be provided by carrying the embankments along the general course of the valley, so as to enclose the windings of the stream without following them, and thus to form not only a reservoir, but a wide and direct channel for the discharge of floods.

The tributary streams which flow into the main streams will in general require branch embankments. Where a main embankment extends for a long distance uninterrupted by a tributary stream, the land protected by it is often divided into portions by means of branch embankments, called "*land arms*," diverging from the main embankment, the object of which is, that, in the event of a breach being made in the main embankment, the inundation may be confined to a limited extent of ground. These "land arms" generally run along the boundaries of separate holdings.

Behind and parallel to each main embankment there runs a "*back drain*," the material dug from which, if suitable, may be used in making the embankment. The use of this back drain is to act not only as a channel for the drainage of the land protected by the embankment, but as a reservoir to collect that drainage when the river is in a state of flood, and its dimensions are to be regulated accordingly. The waters of the back drain are discharged into the river (when its surface is low enough) through a series of pipes traversing the embankment, and having flap-valves opening outwards to prevent the return of water from the river. These valves are made sometimes of iron, sometimes of wood; one of the most efficient consists of an iron grating or perforated plate, covered with a flap of vulcanized indian-rubber. As to the computation of the time required to discharge a given accumulation of water from the back drain through a given outlet, see Article 455, p. 691.

The embankments are to be made of clay rammed in layers one foot deep, or thereabouts. When of moderate height, and not exposed to great pressure, they may have slopes of $1\frac{1}{2}$ to 1 or 2 to 1. When they are liable to be acted upon by a strong current they should be pitched with stone, or otherwise defended like river-banks (Article 469, p. 710): elsewhere they should be covered with sods, and no trees, shrubs, or hedges should be suffered to grow upon them.

484. **Tidal Drainage** is the drainage of lands which are above the low-water-mark of ordinary tides, and either below high-water-mark, or so near that level that their drainage waters can only be discharged in certain states of the tide. Such lands are defended against inundation by the sea by means of embankments, which will be treated of further on.

The best mode of draining a district of this sort is by means of a

canal extending completely through it, which acts alternately as a reservoir and as a channel. The *top-water-level* of the canal is to be fixed so as to give sufficient declivity to the branch drains. Its *low-water-level* will be above that of low-water of neap tides to the extent of 1-15th part of the rise of such tides. The space contained in the canal between those levels is the *reservoir-room;* and inasmuch as the length and depth of that space are fixed, the breadth midway between those levels is to be made sufficient to give reservoir-room for the greatest quantity of drainage water that ever collects during one tide. The depth of the canal must be made at least sufficient to enable the whole of that quantity of water to be discharged in the interval between 1 hour before and 1 hour after low-water, the *mean velocity of outflow* being assumed to be about equal to that due to a declivity of the height between high and low-water-levels in the whole length of the canal, and to its hydraulic mean depth when full up to its middle water-level. The outer end of the canal is to have large floodgates capable of throwing its whole width and depth open at once; or a row of large siphon-pipes, passing over the tidal embankment, and having suitable apparatus for exhausting the air from their summits. (See p. 741.)

485. **Drainage by Pumping** is extensively employed in lands below high-water-mark, especially in Holland. In former times windmills were chiefly used for this purpose, but now they are to a great extent replaced by steam engines. The most economical mode of conducting drainage in this manner is to provide reservoir-room for the greatest floods, and pump constantly at an uniform rate. To provide for the repair of engines, and for accidental stoppages, engines are to be kept in reserve, of power equal to from one-half to the whole of the power of those that are kept at work.

486. **Town Drainage.**—Plans for systems of town drainage require to be on a larger scale, and to have closer contour-lines, than those of any other description of work (See Article 59, p. 96.) The discharge to be provided for is the natural drainage of the basin which the town occupies, added to the water supply artificially brought into the town.

Inasmuch as the rain-fall in towns finds its way into the sewers almost instantly, their dimensions and declivity must be suited to the heaviest rain-fall in a short period. Authorities differ whether that rain-fall is to be estimated at *one inch* or at *half-an-inch* in depth per hour.

The treatment and disposal of the drainage of towns, after it has been collected by means of a system of sewers, involves chemical and physiological questions into which it is impossible to enter in this treatise. (See p. 810.)

487. **Sewers**, or main drains of towns, are underground arched brick conduits, designed, laid out, and constructed according to the principles already explained or referred to in Articles 474, 475, pp. 718 to 720. As to their strength, see Article 297 A, p. 433. The cross-section preferred for them in Britain is an oval, with the small end downwards. In order that men may be able to enter them for purposes of cleansing and repair, no sewer should have a less breadth than 2 feet.

The velocity of the current should be not less than 1 foot per second, or more than about $4\frac{1}{2}$ feet per second.

As to the drainage of streets into the sewers, see Article 417, p. 626. Owing to the quantity of mud that is swept into sewers, they are peculiarly liable to be obstructed by collections of sediment: these are swept away by an operation called *flushing* or *flashing*, which consists in placing a temporary dam of timber above the spot where the deposit is, so as to collect a quantity of water, which is allowed suddenly to escape with great speed in order to scour away the deposit.

As the pipes leading into the sewers from the channels of the streets, and also those from the houses, either are or ought to be "trapped" by means of valves or inverted siphons, so as to prevent the escape of foul gas from the sewers, such gas must have openings provided for its escape, either by building chimneys for the purpose, or by connecting the sewer with existing chimneys. Passages for the admission of fresh air to the sewers are also required, and subterranean entrances with trap-doors to give men access to them. As to the use of "side-trenches" and "subways," see Article 421, pp. 629, 630.

488. **Pipe-Drains.**—The earthenware pipes used for drainage have already been described in Article 477, p.720. In town drainage they are chiefly used for the branch drains leading from houses and from the adjoining ground into the main sewers; and they usually range from 4 inches to 18 inches in diameter, according to the quantity which they are to discharge. It is not advisable in any case to use drain-pipes of less than 4 inches in diameter. They should all be laid, as far as possible, at such declivities as to insure a velocity of flow of $4\frac{1}{2}$ feet per second, in order that the formation of deposit may be impossible; and when their proper levels and declivities have been determined by calculation, great care should be bestowed on seeing that they are accurately laid at those levels and declivities: the smaller the diameter of the pipe, the worse is the effect of any inaccuracy in this respect. Obstructions are most likely to occur at the junctions. The importance of making these either curved or acute-angled has already been mentioned; but even at curved or acute-angled junctions deposits may some-

times take place, and a good safeguard against this, when the levels are such as to render it practicable, is to make the junction in a vertical or transversely inclined, instead of a nearly horizontal plane.

The *inverted siphon air-trap*, for preventing the entrance of foul gas from a sewer into a building through a drain-pipe, is an U-shaped tube, in the lower part of the bend of which water lodges, so as to prevent the passage of gas. To insure the efficiency of this trap, it is essential that the sewer should have chimneys for the escape of gas; otherwise the pressure may become sufficient to enable the gas to force its way past the water in the tube.

Section VI.—*Of Systems of Water Supply.*

489. **Irrigation**. It appears that the supply of water required for the irrigation of a district ranges from ·013 to ·008 of a cubic foot of water per second for each acre irrigated; and this is the *demand* to be provided for by reservoirs, or by the use of weirs to divert water from rivers. (Article 460, p. 699; Article 472, p. 713.) The channels by which the water is distributed are to be carried at the highest levels compatible with the minimum velocity of 1 foot per second, in order that as great an area of land as possible may be commanded by them. Their dimensions and declivity are to be determined by the principles of Article 451, p. 686, and they are to be constructed according to the principles of Section IV. of this chapter, especially Article 474, p. 718. When they run between earthen embankments, as is often the case, each embankment should have a vertical puddle wall in its centre, from 2 to 3 feet thick, and the tops of the embankments should not be less than 4 feet wide.

The method of delivering specified supplies of water from an irrigation canal to holders of land is the following:—A small tank at one side of the canal is supplied through a sluice, and the water in it is kept at a constant level by regulating the opening of that sluice. The water is delivered out of the tank through a square or round orifice of constant size under a constant head. Different quantities of water are delivered by varying the *number* of the orifices, and not their dimensions nor the head which causes their discharge.

490. **Water Supply of Towns—Estimation of Demand as to Quantity.**—The supply of water to towns ranges in extreme cases from about 2 gallons to 600 gallons per inhabitant per day. (Gordon *On Civil Engineering.*) In town water-works executed with a due regard to sufficiency of supply on the one hand and economy of

cost on the other, and with a moderate amount of waste, the following may be regarded as fair estimates of the real daily demand for water per inhabitant amongst inhabitants of different habits as to the quantity of water they consume, (having been verified by the experiments of Mr. J. M. Gale, C.E.)

	Gallons per Day. Least.	Average.	Great
Used for domestic purposes,	7	10	15
Washing streets, extinguishing fires, supplying fountains, &c.,	3	3	3
Trade and manufactures,	7	7	7
Total usefully consumed,	17	20	25
Waste, under careful regulation, say	2	2	2½
Total demand,	19	22	27½

A liberal supply of water has a tendency to increase its use, and at the same time to bring the daily consumption per head amongst different classes of persons more nearly to an equality; so that, with a view to such improvement in the habits of the population, it is advisable in projecting new water-works to take somewhat more than the highest of the preceding estimates of the demand; that is to say, about 30 gallons per head per day, supposing waste of water to be as far as possible prevented.

The quantity of water run to waste, however, frequently exceeds enormously that allowed for in the preceding estimate, through ill-constructed fittings and carelessness. A quantity equal to that used is not uncommon, and in one case, where 7 gallons of water per head per day were actually used, 18 gallons ran to waste. The most effectual means of preventing such waste are, the establishment of a regulation or enactment, that domestic water-fittings shall be executed to the satisfaction of the engineer or manager of the water-works; the carrying out, as far as practicable, of the system of selling water by measure to those who require it for other than ordinary domestic purposes (as to water meters, see Article 459, p. 699); and the prevention of excessive pressure in the service-pipes from which houses are directly supplied.

The preceding statements have reference to the daily demand. Regard must also be had to the hourly demand, which fluctuates very much at different times of the day, chiefly because the inhabitants draw nearly the whole of their supply for domestic purposes during a limited number of hours. It is estimated that the most rapid draught for domestic purposes is at such a rate that,

if kept up continuously, it would exhaust the whole daily supply for these purposes in 8 hours; that is to say, the maximum hourly demand for domestic purposes is *three times* the average hourly demand.

The effect of this on the greatest hourly demand *for all purposes* is to make it in different cases range from twice to $2\frac{1}{8}$ times the average hourly demand.

491. **Estimation of Demand as to Head.**—It is considered that the head of pressure in each of the street mains ought, when the flow is most rapid, to be equivalent to an elevation of about 20 feet above the tops of the adjoining houses, in order that their uppermost stories may be directly supplied, and that it may be possible to throw a jet to the top of the highest building without the aid of a fire-engine.

The required virtual head in various districts of the town being fixed, the virtual declivity from the source to each of those districts is to be made as nearly uniform as circumstances will permit, if pipes are used throughout. Should a conduit be used for part of the distance, and pipes for the remainder, the pipes should have the steeper virtual declivity, and consequently the greater share of the total virtual fall in proportion to their length, in order that they may be smaller than the conduit; because their cost is greater in proportion to their size than that of the conduit. No precise rule can be laid down for this distribution of fall between pipes and conduit; but in some good examples the virtual declivity of the pipes has been made *eight times* as steep as the actual declivity of the conduit. As to the discharging capacity and construction of conduits and pipes, see Articles 450, 451, pp. 684 to 688, and Articles 474 to 478, pp. 718 to 724.

In a town of irregular levels, or of great extent, the same virtual declivity which is required in order to give sufficient head of pressure in the higher parts of the town, or in those more distant from the source, may give excessive pressure in the lower or nearer parts. In such cases the excessive pressure in the branch mains and distributing pipes of the latter districts may be moderated by any convenient means of causing loss of head at their inlets, such as passing the water through small orifices, or loaded valves; the latter being the more accurate method in its working.

492. **Compensation Water** is the supply of water which is secured to the owners and occupiers of land and mills, and other parties interested in the sources from which water is diverted to supply a town, in order that they may not suffer damage by such diversion. It must be at least equal to the supply which was beneficially available for their use before the execution of the water-works, or else they must receive compensation in money for the deficiency.

The only means of enabling a source of water to supply a town, besides providing the landholders with compensation water, according to the preceding principle, is to store in reservoirs and discharge by degrees the flood-waters which previously ran to waste. (See Section III. of this chapter, p. 699.)

In providing the daily supply of compensation water to which the landholders on the course of a stream are entitled, different principles have been followed in different cases. The following are three of them:—

I. To secure them the *average summer discharge, exclusive of floods*, as ascertained by gauging. (As to the distinction between flood discharges and ordinary discharges, see Article 458, p. 698).

II. To give them a proportion fixed by agreement (usually *one-third*, or thereabouts) of the whole water impounded.

In some cases a special arrangement has been come to, by which the landholders, on condition of a certain supply being delivered down the stream during the day, have agreed to a less supply being delivered during the night.

III. To make a special compensation reservoir, receiving the discharge from a certain proportion of the gathering-ground, and to hand it over to the landholders, to be managed under their own control.

The usual method adopted in delivering a fixed daily quantity of water into the natural channel of a stream is to construct a tank in which the water is kept at a fixed level by means of the sluice or sluices through which it is supplied, and let the water flow out of that tank through an outlet or outlets of a fixed area and figure, under a fixed head.

493. **Storage-Works** consist of reservoirs with their appurtenances, as described in Section III. of this chapter. In estimating the extent of gathering-ground and capacity of the reservoirs required, regard must be had to the demand of water for compensation (Article 492), as well as for the supply of the town.

In most cases in which a town is supplied from works of this class, the best economy consists in choosing the sites of the store reservoirs, and designing the conduits and principal main pipes, so as to supply every part of the town by means of the gravitation of the water alone. But exceptional cases sometimes occur, in which a great saving may be effected in capital outlay, and especially in the cost of conduits and pipes, by incurring a comparatively small additional annual expenditure in order to supply some limited district that is highly elevated above the rest of the town by means of a pumping steam engine, instead of giving the conduits and principal main pipes the dimensions required in order to supply that limited district by gravitation.

494. **Springs** in many cases are so variable in their discharge that they can only be classed amongst the sources whose waters require to be stored in a reservoir. But occasionally springs are met with which are the outlets of extensive porous strata, forming underground natural reservoirs that maintain a nearly uniform discharge independently of artificial storage. (See Article 456, p. 696.) When the waters of such springs are diverted from the streams into which they naturally flow in order to supply a town, the ordinary summer flow of those streams must be maintained at its original volume by the aid of the flood-waters of a gathering-ground, stored in a reservoir.

495. **River-Works—Pumping.**—A large river may be used for the supply of a town, independently of storage-works, provided the volume of water brought down by it is at all times so great, that the temporary abstraction of a volume sufficient to supply the town will cause no injury to its navigation, or the interests of the inhabitants of its banks.

The works required in order to supply a town from such a river usually comprise a *weir*, for maintaining part of the river at a nearly constant level (Article 472, p. 713); two or more *settling-ponds*, into which the water is conducted, or if necessary, pumped, or otherwise raised by machinery; filtering apparatus; and a sufficient establishment of pumping engines.

It would be foreign to the plan of the present work to enter into details as to the construction and working of pumping steam engines. The following principles, however, must be stated as specially applicable to their use for the supply of a town.

I. The *effective power* required to be in operation may be computed in *foot-pounds per hour*, by multiplying the *weight* of water to be delivered per hour by the *total head* at the engines in feet; such head being measured from the level of the water in the tank whence the engines draw it, to the virtual elevation required in order to give sufficient head in the town and sufficient virtual declivity in the principal main pipes. To find the *effective horse-power*, divide the effective power in foot-pounds per hour by 1,980,000. The *indicated horse-power* is about *once and a-quarter* the effective horse-power.

II. *Reserve power* should be provided to an amount equal to at least one-half of the working power; for example, of three engines of equal power, two are to be kept at work and the third in reserve.

III. *Air-vessels* and *stand-pipes* are contrivances to prevent the shocks to which the pipes would be exposed by the intermittent action of the pumps, and to maintain an uniform head of pressure and velocity of flow in the pipes.

An air-vessel is an air-tight receiver, usually of cast iron, and of the figure of a cylinder standing vertically, with a hemispherical top and bottom. At its lower end are two openings, an inlet through which water enters from a pump, and an outlet from which the water is discharged along a pipe. Its upper portion contains compressed air, which tends continually to diminish in quantity, partly by leakage and partly by absorption in the water, so that a small supply of air should be forced in from time to time by suitable apparatus. The effect of the air-vessel in moderating fluctuations of pressure is expressed by the following proportion:—

mean volume of air in the vessel : volume of the pump
: : mean head of pressure : greatest fluctuation of the head of pressure.

In some good practical examples, the capacity of the air-vessel is about *fifty times* that of the pump.

A *single stand-pipe* is a vertical cast iron pipe, rising a little higher than the elevation due to the head of pressure, and open at the top. It has at its base an inlet through which it receives water from the pumps, and an outlet or outlets through which it discharges water into the horizontal supply-pipes. Its sectional area varies from once to twice that of its outlets, or thereabouts. It equalizes the pressure and flow even more effectually than an air-vessel, for the rapid entrance of the quantity of water due to one stroke of a pump produces but a slight elevation of the surface of the water in the stand-pipe as compared with its total height.

A *double stand-pipe* has two branches, in one of which the water ascends from the pump, while in the other it descends to the mains: the two branches unite at the top into a vertical stem, which is open above. This construction effects a constant renewal of the water in the stand-pipe.

In estimating the dimensions and speed required for the piston or plunger of a pump that is to deliver a given volume of water in a given time, it is usual to add about *one-fifth* to that volume as an allowance for "*slip;*" that is, water which runs back through the pump-clacks while they are in the act of closing. It appears, however, from experiment, that in the best pumps the slip is not practically appreciable.* (See p. 803).

* The cost of pumping large quantities of water, as ascertained from the accounts of the expenditure of the former Glasgow Water-Works (since superseded by the Loch Katrine Works), during a long series of years, was at the rate of almost exactly 400,000 *gallons raised one foot for a penny;* that is to say, 4,000,000 *foot-pounds of effective work for a penny.*

496. **Wells** may be used as sources for a supply of water, where a water-bearing stratum exists into which they can be sunk. The water in such a stratum has always either an actual or a virtual declivity towards the place where, by the outcrop of the stratum, it makes its escape into a river, or into the sea. Should the water-bearing stratum have its gathering-ground at a high elevation, and should it be covered, in a district far distant from its final outlet, by an impervious stratum, the line of virtual declivity may be above the surface of the ground in that district; so that, on boring or sinking a well through the impervious stratum, the water will spout up in a jet. Such wells are called "Artesian Wells." In other cases the line of virtual or actual declivity is below the surface of the ground, and the water must be raised by pumping (as to which, see the preceding article). (See p. 807.)

The raising of a large quantity of water from a water-bearing stratum has always the effect of depressing the water-level to an extent which cannot be estimated beforehand.

The quantity of water which a water-bearing stratum is capable of yielding may be estimated in the manner explained in Article 456, p. 696, provided the position and extent of its gathering-ground can be ascertained; but that can seldom be done with precision.

In sinking or boring for well water, it is in general advisable to prevent the surface water from mixing with that of the well. This is done, in the case of a bore, by lining it with iron pipes, and in the case of a shaft, by lining it with brickwork laid in cement.

As to boring and shaft-sinking, see Article 187, p. 331, and Article 391, p. 589.

497. The **Purity of Water** is a subject of which the detailed consideration belongs to chemistry and physiology rather than to engineering. The following general principles, however, may be stated.

For purposes of cleansing, cookery, chemistry, and manufactures, the best water is that which approaches nearest to absolute purity. Such is the water which flows from mountain districts, where granite, gneiss, and slate prevail. Such water usually contains a large quantity of diffused oxygen and carbonic acid. It is the most wholesome for drinking, and the most agreeable to those whose taste does not prefer a certain admixture of earthy salts.

The most common mineral impurities of water are salts of lime and iron, which injure it for all purposes except drinking. Salts of lime, especially the bicarbonate, are the principal causes of the property called "hardness." The bicarbonate of lime can be removed by adding to the water as much lime-water as contains a quantity of lime equal to that already contained in the bicarbonate

of lime present. The additional lime thus added combines with one-half of the carbonic acid, thus becoming chalk itself, and reducing the bicarbonate to chalk also; and the chalk, being insoluble, settles, and leaves the water softened. This is Dr. Clarke's process of softening water. The *degrees of hardness* of a specimen of water means the number of grains of chalk which the lime held in solution in a gallon of the water (or 70,000 grains) is capable of forming. Water of less than 5 degrees of hardness may be considered as comparatively soft; that of 12 or 13, as decidedly hard.

The waters collected directly from gathering-grounds are usually the softest, those of rivers harder, those of springs and wells hardest of all.

The drainage waters of cultivated and populous districts, and above all, those of towns and their neighbourhood, are to be avoided, as containing organic matter in the act of decomposition, and being therefore unwholesome, and sometimes highly dangerous.

The taste and smell of a person accustomed to drink pure water and breathe pure air may in general be relied upon for the detection of the presence of impurities in water, though not of their nature or amount; but in persons who have for some time habitually drunk impure water and breathed a foul atmosphere those senses become blunted.

The colouring matter of peat moss, which is a compound of carbon with oxygen and hydrogen, unfits water for many manufacturing purposes. It does not render it unfit for drinking, unless present in considerable quantity, when it produces an unpleasant flatness of taste; but whether that substance is unwholesome or not has not been ascertained. Its appearance is strongly objected to by the inhabitants of most towns. Long exposure to light and air destroys it, probably by oxidating its carbon.

The long-continued action of oxygen decomposes and destroys organic matter in water, and is the principal means of purifying originally impure water. In store reservoirs the presence of a moderate quantity of living plants is favourable to purity of the water, provided there are also animals enough to consume them, so that they may not die and decompose, and that a proper balance is kept up amongst animals of different kinds. The destruction of the fish in a reservoir has been known to lead to an excessive multiplication of the small crustaceous animals upon which the fish had fed, to such an extent that the water acquired a nauseous flavour from the oil which those minute creatures contained. The only remedy was to re-stock the reservoir with fish.*

* This case was examined into and reported upon, and the remedy discovered, by Dr. H. D. Rogers.

Shallow reservoirs are unfavourable to purity, because the warmth of the water produced by the sun's heat encourages the growth of an excessive quantity of vegetation, most of which dies and decomposes.

On the subject of the purity of water, see Dr. R. Angus Smith's "Report on the Air and Water of Towns," in the *Reports of the British Association* for 1851. (See also Appendix.)

498. **Settling and Filtration.**—A store reservoir generally answers the purpose of a settling-pond also, to clear the water of earthy matter held in suspension. Water pumped from a river generally requires to rest for a time in a settling-pond.

The water both of rivers and of gathering-grounds in most cases requires to be filtered. A filter-bed for that purpose consists of a tank about 5 feet deep, having a paved bottom, covered with open-jointed tubular drains leading into a central culvert; the drains are covered with a layer of gravel about 3 feet deep, and that with a layer of sand 2 or 3 feet deep. The water is delivered upon the upper surface of the sand very slowly and uniformly; it gradually descends, and is collected by the drains into the central culvert. The area of the filter should be such that the water to be filtered may not descend vertically with more than a certain speed; for the whole efficiency of the filtering process depends on its slowness. The speed of vertical descent recommended by the best authorities is *six inches an hour;* in some cases a speed as high as *one foot an hour* has been used.

There should be a sufficient number of filter-beds to enable some to be cleansed whilst others are in use. The cleansing is performed by scraping from the surface of the sand a thin layer, in which all the dirt collects.

It appears that proper filtration not merely removes mechanical impurities from the water, but even organic impurities, by causing their oxidation. (See pp. 792 and 805.)

499. **Distributing-Basins or Town Reservoirs.**—It has been explained in Article 490, p. 730, that the *greatest hourly demand* for water is about double of the *average hourly demand;* from which it follows, that the pipe or conduit which *directly* supplies a given town, or part of a town, must have about double the discharging capacity that it would require if the hourly demand were uniform.

The great additional expense which this would cause in the principal conduits and main pipes is saved by the use of *distributing-basins* or *town reservoirs.*

A distributing-basin for a given district is a small reservoir, capable of containing a volume of water *at least* equal to the whole excess of the demand for water during those hours of the day when

such demand exceeds the average rate above a supply during the same time at the average rate. The smallest capacity which will enable a distributing-basin to fulfil that condition is about one-half of the daily demand of the district to which it belongs; but to provide for unforeseen contingencies, it may be made to contain a whole day's demand, or even more. It is supplied with water at an uniform rate, by a principal main pipe, which thus only needs to be made capable of supplying the average hourly demand, the distributing-pipes along requiring to be adapted to the greatest hourly demand. During the night, when the supply exceeds the demand, the water accumulates in the distributing-basin; during the day, when the demand exceeds the supply, that accumulated water is expended.

The area of a distributing-basin should be such, that the variation of its water-level may not cause an inconvenient variation of the head of pressure in the pipes, nor in their virtual declivity.

It may be built and paved with masonry or brickwork lined with cement, in which case the stability of its walls will depend on the principles cited in Article 465, p. 707; or it may be made of rectangular cast iron plates, flanged and bolted together, the opposite sides of the reservoir being tied together by means of wrought iron rods, to enable them to resist the pressure. The figure in plan will in general be regulated by that of the site; but should the engineer be free to choose any figure, the circular figure is obviously the best.

The elevation of the site should be such as to command the district to be supplied from the basin, according to the principles of Article 491, p. 732, and it should be as near that district as possible.

Every distributing-basin should be roofed, that the water may be protected against heat, frost, and the dust and soot which float in the air of populous districts. The most efficient protection against heat and frost is that given by a vaulted roof of masonry or brick, covered with asphaltic concrete to exclude surface water, and with two or three feet of soil, and a layer of turf.

When water is brought to a city from a great distance, it may be useful to construct in the neighbourhood of the city (should the ground afford a suitable site), a large town reservoir or auxiliary store reservoir, capable of holding a store of water for about a month's demand, to be used in the event of an accident happening to the more distant part of the main conduit, until the damage is repaired. From that reservoir to the town the main pipes may form a double line, so that in the event of a failure of one line, a supply, although a diminished one, may be conveyed through the other line until the first line is repaired. The construction of

such an auxiliary store reservoir will in general be similar to that of the reservoirs described in Section III. of this chapter.

500. **Distributing-Pipes** must be adapted to the *greatest* hourly demand for water, and to the requisite head in the streets, as already explained in Articles 490 and 491, pp. 730 to 733. In large cities the total length of distributing-pipes required is about a mile for every 2,000 or 3,000 inhabitants. The smaller the town, the smaller in general is the *proportionate* extent of distributing-pipes required.

The distributing-pipes which are laid along the street are classed as *mains* and *service-pipes;* the chief distinction being, that a main either conveys, or is capable of conveying, water along a street to some place beyond it; while a service-pipe is a branch diverging from a main, in order to supply a single or double row of buildings. In wide streets, and in those of great traffic, it is best to have two service-pipes, one for each side, in order that they may be laid so as to be accessible without interrupting the traffic of the street (see Article 421, p. 629), and in order that the house water-pipes may be as short as possible, and may lie as little as possible under the carriage-way.

When a general rate of virtual declivity has been fixed for the distributing-pipes of a town or of a district of a town, and the diameters of the more important mains have been computed by the proper formula, those of all branch mains and service-pipes are easily deduced from them by the rule, that, with equal virtual declivities, the diameters of pipes are to be proportional to the *squares of the fifth roots* of the quantities of water that they are to convey.

When a pipe of uniform diameter has a series of branches diverging from it, so that the flow of water through it becomes less and less at an uniform rate, until the pipe terminates at a "*dead end*," the virtual declivity goes on diminishing, being proportional to the *square of the distance from the dead end;* the excess of the head at any point above the head at the dead end is proportional to the *cube of the distance from the dead end;* and the total virtual fall, from the commencement of the pipe to the dead end, is *one-third* of what it would have been had the whole quantity of water flowed along the pipe without diverging into branch pipes.

All *dead ends* of pipes should be provided with *scouring-valves,* which should be opened from time to time to prevent the accumulation of deposit there. Pipes should be laid out and connected with each other so as to have as few dead ends as possible; and with that view it is desirable that service-pipes should, if practicable, be connected at both ends with mains.

The use of loaded valves to moderate pressure has already been mentioned in Article 491, p. 732.

The system called that of *constant service*, according to which all distributing-pipes are kept charged with water at all times, is the best, not only for the convenience of the inhabitants, but also for the durability of the pipes, and for the purity of the water, for pipes, when alternately wet and dry, tend to rust; and when emptied of water, they are liable to collect rust, dust, coal gas, and the effluvia of neighbouring sewers, which are absorbed by the water on its re-admission. In order, however, that the system of constant service may be carried out with efficiency and economy, it is necessary that the diameters of the pipes should be carefully adapted to their discharges, and to the elevation of the district which they are to supply, and that the town should be sufficiently provided with town reservoirs. When these conditions are not fulfilled, it may be indispensable to practise the system of *intermittent service*, especially as regards elevated districts, that is to say, to supply certain districts in succession, during certain hours of the day. The adoption of this system makes it necessary for the inhabitants to have cisterns in their houses for the purpose of holding the daily store of water. In the poorer districts of towns, it is often advisable to have one large tank for a group of small houses, instead of a cistern in each house; the tank may be under the control of the water-work officials, and may be filled once a day, and the householders may be supplied from it through small pipes constantly charged, and may thus have the convenience of constant service although the supply to the tank is intermittent.

500A. On the subject of the collection, conveyance, and distribution of water generally, special reference may be made to Fanning, *Water Supply;* Humber, *Water Works;* Reports of the Government Hydraulic Engineer, Queensland; Latham, *Sanitary Engineering;* Robinson, *Hydraulic Power*, 2nd Ed., 1893; Crimp, *Sewage Disposal Works*, 2nd Ed., 1894; Gale, "Loch Katrine Water Works," *Trans. Inst. Engineers and Shipbuilders in Scotland*, vols. vii. and xxxviii.; Frankland, "Water Purification," *Minutes Proceed. Inst. C.E.*, vols lxxxv. and cxxvii; and on that of the water supply of towns, to the *Parliamentary Reports* on the supply of water to the Metropolis, and the *Reports* of the Board of Health on the same subject.

ADDENDUM to Article 484, p 728 —**Siphons for Tidal Drainage.**—The waters of the Middle-Level Drainage Canal are discharged over the top of an embankment through sixteen parallel siphons, each of $3\frac{1}{2}$ feet bore and $\frac{1}{2}$ inch thick. The summits of the siphons are 20 feet above, and their lower ends $1\frac{1}{2}$ foot below, low water of spring-tides. They have flap-valves, opening down stream, at both ends; the lower valve can be made fast with a bridle when required. The air is exhausted from their summits, when required, by an air-pump having three cylinders of 15 inches diameter and 18 inches stroke, driven by a high-pressure steam engine of ten horse power. The floor of the canal at the inlets and outlets is protected by a wooden apron. (J. Hawkshaw, C.E., F.R.S., in the *Proceedings of the Institution of Civil Engineers*, April, 1863.)

CHAPTER IV.

OF WORKS OF INLAND NAVIGATION.

Section I.—*Of Canals.*

501. **Canals Classed - Selection of Line and Levels.**—Canals may be divided into three classes—

I. *Level Canals*, or *Ditch Canals*, consisting of one *reach* or *pond*, which is at the same level throughout. The most economical course for a canal of this sort is obviously one which nearly follows a contour-line, except where opportunities occur of saving expense by crossing a ridge or a valley so as to avoid a long circuit.

II. *Lateral Canals*, which connect two places in the same valley, and in which, therefore, there is no summit level, the fall taking place in one direction only. A lateral canal is divided into a series of level reaches or ponds, connected by sudden changes of level, at which there are either single locks or flights of locks, or some other means of transferring boats from one level to another. The "lift" of a single lock ranges from 2 feet to 12 feet, and is most commonly 8 or 9 feet. Each level reach is to be laid out on the same principles with a level canal. In fixing the lengths of the reaches and the positions of the locks, the engineer should have regard to the fact that economy of water is promoted by distributing a given fall amongst single locks with reaches between them, rather than concentrating the whole fall at one flight of locks.

III. *Canals with Summits* have to be laid out with a view to economy of works at the passes between one valley and another, and with a view also to the obtaining of sufficient supplies of water at the summit reaches. The subject of the supply of water to canals will be considered further on. (See pp. 807 and 809.)

502. **Form and Dimensions of Water-way.**—Although, for the sake of saving expense in aqueducts and bridges, short portions of a canal may be made wide enough for the passage of one boat only, the general width ought to be sufficient to allow two boats to pass each other easily. The depth of water and sectional area of water-way should be such as not to cause any material increase of the resistance to the motion of the boat beyond what it would encounter in open water. The following are the general rules which fulfil these conditions:—

Least Breadth at Bottom = 2 × greatest breadth of a boat.
Least Depth of Water = 1½ foot + greatest draught of a boat.
Least Area of Water-way = 6 × greatest midship section of a boat.

The bottom of the water-way is flat. The sides, when of earth (which is generally the case), should not be steeper than 1½ to 1; when of masonry, they may be vertical; but, in that case, about 2 feet additional width at the bottom must be given to enable boats to clear each other, and if the length traversed between vertical sides is great, as much more additional width as may be necessary in order to give sufficient sectional area.

The customary dimensions of canal-boats have been fixed with a view to horse-haulage. The most economical use of horse-power on a canal is to draw heavy boats at low speeds. The heaviest boat that one horse can draw at a speed of from 2 to 2½ miles an hour weighs, with its cargo, about 105 tons, is about 70 feet long and 12 feet broad, and draws about 4½ feet of water when fully loaded. Smaller boats, which a horse can draw at 3½ or 4 miles an hour, are of about the same length, 6 or 7 feet broad, and draw about 2½ feet of water.

Boats of the greater breadth above-mentioned can easily be adapted to the various methods of propulsion by steam, whether by means of the screw propeller or the warping chain, or fixed engines and endless wire ropes. (See Appendix.)

Ordinary canals are suited to boats such as the above. A larger class of canals are suited to sea-going vessels.

The following are examples of ordinary dimensions of canals :—

	Breadth. at Bottom.	Breadth. at Top-water.	Depth. of Water.
Small canal,.......	25 feet, ...	45 to 50 feet, ...	6 feet.
Ordinary canal,...	40 ,, ...	70 to 80 ,, ..	10 ,,
Large canal,.......	80 ,, ...	140 to 160 ,, ...	20 ,,

503. **Construction of a Canal.**—The least expensive parts of a canal are those in which the upper part of the water-way is contained between two embankments, and the lower part in a cutting, the earth dug from which, together with that dug from the side-drains at the foot of the outer slopes, is just sufficient to form the embankments.

All canal embankments should be formed and rammed in thin layers. (Article 203, p. 341.) The width of the embankment which carries the towing-path is usually about 12 feet at the top; that of the opposite embankment at least 4 feet, and sometimes 6 feet. Each embankment has a vertical puddle wall in its centre from 2 to 3 feet thick. (See Appendix, p. 807.)

In cutting, there should be a bench or berm of 12 or 14 feet wide, at one side, for the towing-path, and on the opposite side a bench about 3 or 4 feet wide at the same level. At the feet of the slopes, which terminate at those benches, there are a pair of side-drains, as described in Article 193, p. 335. These side-drains discharge their water at intervals into the canal through tubes.

The surface of the towing-path is usually about 2 feet above the water-level. It is made to slope slightly in a direction away from the canal, in order to give a better foot-hold for the horses, as they draw in an oblique direction.

The slopes are to be pitched with dry stone from 6 to 9 inches thick. For the wash of large boats the pitching should be increased.

Occasionally it may be necessary to line a canal with concrete, or to face the sides with rows of sheet-piling, in order to retain the water.

Natural water-courses are to be carried below the canal by means of bridges and culverts, and, if necessary, by inverted siphons of masonry or iron. Where such water-courses are above the level of the canal, their waters may be partly used for supplying it; but means should be provided for carrying such waters wholly across the canal when required.

Each reach of a canal should be provided with waste-weirs in suitable positions, to prevent its waters from rising to too high a level; also with sluices, through which it may be wholly emptied of water for purposes of repair; and in a reach longer than two miles, or thereabouts, there may be stop-gates at intervals, so that one division of the reach may be emptied at a time, if necessary. The rectangular channel under a bridge or over an aqueduct is a suitable place for such gates.

Leaks in canals may sometimes be stopped by shaking loose sand, clay, lime, chaff, &c., into the water. The particles are carried into the leaks, which they eventually choke by their accumulation.

504. **Canal Aqueducts and Fixed Bridges.**—A canal aqueduct, like the aqueducts for conduits already mentioned in Article 476, p. 720, is a bridge supporting a water-channel. The trough or channel, for economy's sake, is usually made wide enough for one boat only. Its bottom is flat, or nearly so; its sides vertical or slightly battering. In aqueducts of masonry, the total thickness of material, from the side of the trough to the face of the spandril-wall, is usually 4 feet at least at the side furthest from the towing-path; at the towing-path side it is sufficient for a towing-path of from 6 to 10 feet wide, and a parapet from 15 to 18 inches thick.

In Telford's cast iron aqueduct, known as Pont-y-Cysylte, the channel is a rectangular trough of cast iron, supported on cast iron segmental arched ribs of 45 feet span. The trough is of the whole

width of the bridge, about 12 feet, and the towing-path, 5 feet 8 inches wide, covers part of the trough.

The principle of the *suspension bridge* is peculiarly well adapted to aqueducts, because, as each boat displaces its own weight of water, the only disturbance of the uniform distribution of the load is that arising from the passage of men and horses along the towing-path. An aqueduct of this sort, designed by Mr. Roebling, with seven spans of 160 feet, carries a canal 16 feet wide and 8 feet deep, over the Alleghany River at Pittsburg.

Fixed bridges over canals require no special explanation, except to state that, in the older examples of them, the water-way is contracted so as to admit one boat only, and the towing-path is only 6 feet wide, or thereabouts, the headroom over it being about 10 feet. Sometimes the archway admits the water-channel alone, while the towing-path ascends to the approach of the bridge and descends again, the tow-rope being cast loose while the horse passes over. As to bridges for carrying railways over canals, see Article 436, p. 663.

Tunnels for canals usually have the water-way and towing-path contracted as already described; and sometimes the towing-path is dispensed with, the boats being pushed through by means of poles, or by the hands and feet of the boatmen, with the aid of notches in the brickwork, or by means of the various methods of steam propulsion.

505. **Moveable Bridges** cross a canal near its water-level are made of timber or of iron, and are capable of being opened so as to leave the navigation clear, and closed so as to form a passage for a road or railway by one or other of five kinds of movement, viz., I. By turning about a horizontal axis; II. By turning about a vertical axis; III. By rolling horizontally; IV. By lifting vertically; V. By floating in the canal. As regards the adaptation of the strength and stiffness of a moveable bridge to the greatest load which it has to bear when closed, it differs in no respect from a fixed bridge. But, besides having the strength and stiffness required in a fixed bridge, it must fulfil some other conditions, which are as follows:—If it turns about an axis, it must be so balanced that its centre of gravity shall always lie in that axis; if it rolls backwards and forwards it must be so balanced that its centre of gravity shall always lie over the base or platform on which it rolls: in either of those cases it must have strength sufficient to support safely the *overhanging* part of its own structure, when deprived of direct support; if it is lifted vertically, it must be counterpoised; and if it is carried by a pontoon or float, that float must displace a mass of water equal in weight to the bridge, and must have sufficient stability. (See p. 808.)

I. A bridge which turns about a horizontal axis near an end of its span is called a *draw-bridge*. It is opened by being raised into a vertical position by means of a pinion driving a toothed sector. It is best suited for small spans.

II. A bridge which turns about a vertical axis is called a *swing-bridge*. Its principal parts are as follows:—

A pier of masonry or iron, supporting a circular base-plate of a diameter equal, or nearly equal, to the breadth of the bridge. That base-plate has a pivot in the centre, and a circular race or track for rollers round the circumference, as in a railway turntable:

A roller frame turning about the central pivot, with a set of conical rollers resting on the race:

A circular revolving platform resting on the pivot and rollers:

A toothed arc fixed to the revolving platform, with suitable wheel-work for giving it motion:

A set of parallel girders, resting on and fastened to the revolving platform, of the strength and stiffness required by the principles already stated, and supporting a roadway.

The ends of the superstructure are bounded by arcs of circles, described about the axis of motion, and the ends of the roadway of the approaches must be formed to fit them.*

III. A *rolling bridge* has a strong frame, supported by wheels upon a line of rails, and having an overhanging portion sufficient to span the water-way. When closed, by being rolled forward, the rolling frame leaves a gap between its platform and that of one of the approaches, which gap is filled by rolling in another rolling frame that moves sideways. The latter rolling frame is rolled out of the way before opening the bridge.

IV. A *lifting bridge* is hung by the four corners to four chains, which pass over pullies, and have counterpoises at their other ends.

V. A *floating swing-bridge* rests on a caisson or pontoon: it is opened and closed by means of chains and windlasses, and, when open, lies in a recess in the side of the canal made to receive it. The pontoon, being made of sheet iron, is so designed as to act as a tubular girder when the bridge is closed.

506. **Canal Locks.**—Figs. 290, 291, and 292, show the general arrangement of the parts of a canal lock. Fig. 290 is a longitudinal section, fig. 291, a plan, and fig. 292 a cross-section, looking upwards.

* For an example of a swing-bridge on a great scale, reference may be made to one planned by Mr. Hemans and constructed by Messrs. Fairbairn, which carries the Midland Great Western Railway of Ireland over the entrance to Lough Atalia. It has two spans of 60 feet each, and is balanced on a central pier of 84 feet diameter. It is described in detail in Mr. Humber's work *On Iron Bridges*.

A is the ***lock-chamber;*** ***a, a,*** its side walls; **E**, its floor, or invert.

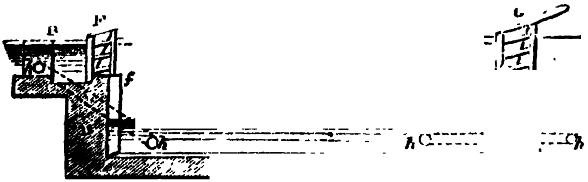


Fig. 290.

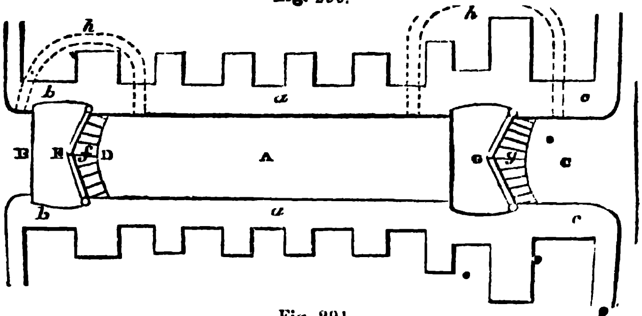


Fig. 291.

Its clear length should be at least equal to that of the longest vessel used on the canal, including the rudder; its clear breadth, one foot more than the greatest breadth of a vessel; its greatest depth of water should be = $1\frac{1}{2}$ foot + greatest draught of a vessel + lift of the lock. Its depth from the cope of the side walls to the bottom may be about 2 feet more.

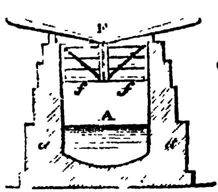


Fig. 292.

The side walls and floor are recessed to admit of the opening of the "tail-gates."

The floor is level with the bottom of the lower of the two ponds to be connected.

B is the *head-bay*, with its side walls and floor, which are recessed to admit of the opening of the "head-gates." The side-walls end in curved wings. The floor is level with the bottom of the upper pond.

C, the *tail bay*, with its side walls and floor. The side walls end in curved wings: the floor in a dry stone pitching or *apron*.

D, the ***lift-wall***, which is usually built like a horizontal arch.

F, the ***head-gates***, whose lower edges, when shut, press against the ***head mitre-sill, f.***

G, the *tail-gates*, whose lower edges, when shut, press against the *tail mitre-sill*, g.

The older locks are filled and emptied through sluices in their head and tail-gates; but now the more general practice is to use for that purpose culverts extending the whole length of the lock, with openings into the lock chamber, through which the water can be either admitted or withdrawn.

The cylindrical recesses in which the gates are hinged are called the *hollow quoins*.

The following parts of a lock are usually of ashlar:—The quoins, hollow quoins, cope, recesses for the gates (or "gate-chambers"), and mitre-sills.

The mitre-sills are sometimes faced with wood, to enable them the better to withstand the blows which they receive from the gates, and to make a tighter joint.

The floor of the lock is sometimes made of cast iron. (See Article 400, p. 601.)

The gates are made of timber or of iron, and each of them consists of the following principal parts:—

The *heel-post*, about the axis of which the gate turns. This post is cylindrical on the side next the hollow quoins, which it exactly fits when the gate is shut. It is advisable to make it slightly eccentric, so that when the gate is opened, it may cease to rub on the hollow quoins. At its lower end it rests on a pivot, and its upper end turns in a circular collar, which is strongly anchored back to the masonry of the side walls:

The *mitre-post*, forming the outer edge of the frame of the gate, which, when the gate is shut, abuts against and makes a tight joint with the mitre-post of the opposite leaf:

The *cross-pieces*, which extend horizontally between the heel-post and mitre-post:

The *cleading* or covering, which may consist of timber planking or iron plates. When it consists of planks, they run either vertically or diagonally:

The *diagonal bracing*, which, in its simplest form, may consist either of a timber strut extending from the bottom of the heel-post to the top of the mitre-post, or of an iron tie-bar extending from the top of the heel-post to the bottom of the mitre-post.

The gates shown in the sketch are provided with *balance-bars*. A balance-bar is bolted to the top of the mitre-post, slopes slightly upwards, and crosses over the top of the heel-post, which is mortised into it, and has a long and heavy overhanging end, which acts as a counterpoise to bring the centre of gravity of the gate near the heel-post, and as a lever to open and shut it by.

Sometimes the balance-bar is dispensed with, and each gate has

one or more *rollers* under its lowest cross-bar, to assist the pivot in supporting its weight. Each of those rollers runs upon a quadrantal iron rail on the floor of the gate-chamber. This mode of construction is almost always adopted in large and heavy gates that require chains and windlasses to open and shut them.

The following are some of the ordinary dimensions and proportions of locks, in addition to those already stated:—

The mitre-sills rise from 6 to 9 inches above the floor:

Versed-sine of mitre-sill, from $\frac{1}{7}$ to $\frac{1}{5}$ of breadth of lock:

Clearance in depth of the recesses for the gates, $\frac{1}{10}$ of thickness of gate; clearance in length, $\frac{1}{7}$ of length of gate:

Least thickness of the side walls at the top, about 4 feet. Greatest thickness at the base, fixed according to the principles of the stability of walls, usually from $\frac{1}{4}$ to $\frac{1}{2}$ of the height:

Length of side walls of head-bay above gate-chamber, about $\frac{1}{3}$ of breadth of lock:

Large counterforts opposite hollow quoins to have stability enough to withstand the calculated *transverse thrust* of the gates.

The *longitudinal thrust* of the head-gates is borne by the side walls of the lock-chamber; that of the tail-gates by the side walls of the tail-bay. To give the latter walls sufficient stability, the rule is to make their length as follows:—

$$\text{Breadth of lock} \times \text{greatest depth of water} \div 15 \text{ feet.}$$

Versed-sine of lift-wall, from 1-12th to 1-7th of breadth of lock.

Floor of head-bay: least thickness, from 10 inches to 14 inches.

Floor of lock-chamber: versed-sine, about 1-15th of breadth; thickness, from 1-15th to 1-3rd of breadth, according to the nature of the foundation.

Foundations of various kinds have been sufficiently explained. It has only to be added that, when a lock is founded on a timber platform, longitudinal pieces of timber extending along the whole length of the foundation are to be avoided, lest they guide streams of water along their sides; that transverse trenches under the foundation, filled with hydraulic concrete, are a good means of preventing leakage; and that, in porous soils, the whole space behind the lift-wall and under the floor of the head-bay may be filled with a mass of concrete.

Length of apron from 15 to 30 feet.

The dimensions of the different parts of the gates are to be computed according to the principles of the strength of materials. It appears that the factor of safety in many actual lock-gates is as low as 3 or 4. This can only be sufficient by reason of the perfect steadiness of the load.

507. Inclined Planes on Canals.—To save the time and water

expended in shifting boats from one level to another by means of locks, inclined planes are used on some canals. Their general arrangement is as follows:—The upper and lower reach of the canal, at the places which are to be connected by inclined planes, are deepened sufficiently to admit of the introduction of water-tight iron caissons, or moveable tanks, under the boats. Two parallel lines of rails start from the bottom of the lower reach, ascend an inclined plane up to a summit a little above the water-level of the upper reach, and then descend down a short inclined plane to the bottom of the upper reach. There are two caissons, or moveable tanks on wheels, each holding water enough to float a boat. One of these caissons runs on each line of rails; and they are so connected, by means of a chain, or of a wire rope, running on moveable pullies, that when one descends the other ascends. These caissons balance each other at all times when both are on the long incline, because the boats, light or heavy, which they contain, displace exactly their own weight of water. There is a short period when both caissons are in the act of coming out of the water, one at the upper and the other at the lower reach, when the balance is not maintained; and, in order to supply the power required at that time, and to overcome friction, a steam engine drives the main pully, as in the case of fixed-engine planes balance is not maintained; and, in order to supply the power required at that time, and to overcome friction, a steam engine drives the main pully, as in the case of fixed-engine planes on railways.

On some canals vertical lifts with caissons are used instead of inclined planes, notably at Anderton, near Norwich; also in France and Belgium. The following are the leading dimensions:—

Name of Lift,	Anderton.	Fontuettes.	La Louvière.
Lift, ...	50 ft. 2 ins.	43 ft.	50 ft. 6 ins.
Length of box between gates,.... ..	73 ft. 9 ins.	132 ft. 10½ ins.	141 ft. 7 ins.
Width of box, ...	15 ft. 3 ins.	17 ft.	18 ft. 4 ins.
Depth of water in box, ...	4 ft. 5 ins.	6 ft. 6¾ ins.	8 ft. 6 ins.
Diameter of rams, ...	2 ft. 11·4 ins.	6 ft. 6¾ ins.	6 ft. 6¾ ins.
Weight to be lifted, ...	250 tons.	770 tons.	1100 tons.
Displacement of largest boat, ..	100 tons.	300 tons.	400 tons.

In this method of overcoming the difference of level on a canal route, the boats are floated in iron troughs; these are raised or lowered by hydraulic power.

Recently, what is called a "Pneumatic Balance Canal Lock" has been introduced on the Erie Canal in America, where there is a lift of over 60 feet. Two steel chambers for ascending and descending boats are used. The lock chamber containing the boat is floated on compressed air held in the lower part of the casing.

508. **Water Supply of Canals.**—Canals are supplied with water from gathering-grounds, springs, rivers, and wells, by the aid of reservoirs and conduits; and their supply involves the same questions of rain-fall, demand, compensation, &c., which have already been treated of in Chapter II. of this Part.

The *demand* for water, in the case of a canal, may be estimated as follows:—

I. *Waste of Water* by leakage of the channel, repairs, and evaporation, *per day* = area of surface of the canal × $\frac{1}{6}$ of a foot, nearly.

II. *Current* from the higher towards the lower reaches, produced by leakage at the lock gates, *per day*, from 10,000 to 20,000 cubic feet, in ordinary cases.

III. *Lockage*, or expenditure of water in passing boats from one level to another.

Let L denote a *lockful* of water; that is, the volume contained in the lock-chamber, between the upper and lower water-levels.

B, the volume displaced by a boat.

Then the quantities of water discharged from the upper pond, at a lock or a flight of locks, under various circumstances, are shown in the following tables. The sign − prefixed to a quantity of water denotes that it is displaced *from the lock into the upper pond.*

SINGLE LOCK.	Lock found,	Water discharged.	Lock left,
One boat descending.........	empty,.	L − B	empty.
	full, . .	− B	
One boat ascending,.........	empty or full,. .	L + B	full.
2 *n* boats, descending and ascending alternately,	descending full, ascending empty	*n* L	descending empty. ascending full.
Train of *n* boats descending,	empty,...........	*n* L − *n* B	empty.
" " "	full, ...	(*n* − 1) L − *n* B	
Train of *n* boats ascending,	empty or full,...	*n* L + *n* B	full.
Two trains, each of *n* boats, the first descending, the second ascending,	full, ...	(2 *n* − 1) L	full.

FLIGHT OF *m* LOCKS.	Locks found,	Water discharged.	Locks left,
One boat descending,.........	empty,	L − B	empty.
" "	full, .	− B	
One boat ascending, ...	empty, ...	*m* L + B	full.
	full, .	L + B	
2 *n* boats, descending and ascending alternately,	descending full, ascending empty	*m n* L	descending empty. ascending full.
Train of *n* boats descending,	empty,	L − *n* B	empty.
" " "	full, .	(*n* − 1) L − *n* B	
Train of *n* boats ascending,	empty,...........	(*m*+*n* − 1) L + *n* B	full.
" " "	full, ...	*n* L + *n* B	
Two trains, each of *n* boats, the first descending, the second ascending,	full, ...	(*m* + 2 *n* − 2) L	full.

From these calculations it appears, as has been already stated, that single locks are more favourable to economy of water than flights of locks; that at a single lock single boats ascending and descending alternately cause less expenditure of water than equal numbers of boats in trains; and that, on the other hand, at a flight

of locks, boats in trains cause less expenditure of water than equal numbers of boats ascending and descending alternately.

For this reason, when a long flight of locks is unavoidable, it is usual to make it double; that is, to have two similar flights side by side—using one exclusively for ascending boats and the other exclusively for descending boats.

Water may be saved at flights of locks by the aid of *side ponds* (sometimes called "lateral reservoirs"). The use of a side pond is to keep for future use a certain portion of the water discharged from a lock, when the locks below it in the flight are full, which water would otherwise be wholly discharged into the lower reach. Let a be the horizontal area of a lock-chamber, A that of its side pond; then the volume of water so saved is—

$$L A \div (A + a).$$

SECTION II.—*Of River Navigation.*

509. An **Open River** is one in which the water is left to take a continuous declivity, being uninterrupted by weirs. On the subject of such streams little has here to be added to what has already been stated in articles 467 to 471, pp. 707 to 713. The towing-path required, if horse haulage is to be employed, is similar to that of a canal.

The effect of the current of the stream on the load which one horse is able to draw against it at a walk may be roughly estimated as follows:—

$$\text{Load drawn against current} = \text{load drawn in still water} \times \left(\frac{3{\cdot}6}{3{\cdot}6 + v}\right)^2$$

v being the velocity of the current in feet per second.

It would be foreign to the subject of this work to discuss the principles of the propulsion of vessels by steam and sails.

510. A **Canalized River** is one in which a series of ponds or reaches, with a greater depth of water and a slower current than the river in its natural state, have been produced by means of weirs. The construction and effect of weirs have been explained in Article 472, p. 713, and the previous articles there referred to.

Each weir on a navigable river requires to be traversed by a lock for the passage of vessels, the most convenient place for which is usually near one end of the weir, next the bank where the towing-path is. The head gates are of less height than the tail gates by the lift of the lock.

511. **Movable Bridges over Rivers** are identical in principle with those over canals, and differ from them only in being of greater size. Examples of them have already been cited in Article 505, p. 745.

CHAPTER IV.

OF TIDAL AND COAST WORKS.

SECTION I.—*Of Waves and Tides.*

512. **Motion of Ordinary Waves.**—The following description of wave-motion in water is founded chiefly on the theoretical investigations of Mr. Airy and others, and the observations of the Messrs. Weber and of Mr. Scott Russell, with a few additions founded on later researches.

Rolling waves in water are propagated horizontally; the motion of each particle takes place in a vertical plane, parallel to the direction of propagation; the path or orbit described by each particle is approximately elliptic (see fig. 293), and in water of uniform depth the longer axis of the elliptic orbit is horizontal, and the shorter vertical; the centre of that orbit lies a little above the position that the particle occupies when the water is undisturbed; when at the top of its orbit, the particle moves *forwards* as regards the direction of propagation; when at the bottom, backwards, as shown by the curved arrows in fig. 293, in which the straight feathered arrow denotes the direction of propagation.

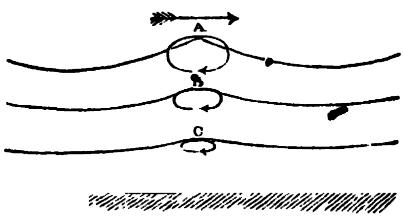


Fig. 293.

The particles at the surface of the water describe the largest orbits; the extent of the motion, both horizontally and vertically, diminishes as the depth below the surface increases; but that of the vertical motion more rapidly than that of the horizontal motion, so that the deeper a particle is situated the more flattened is its orbit, as indicated at A, B, and C; a particle in contact with the bottom moves backwards and forwards in a horizontal straight line, as at D.

In water that is deep, as compared with the length of a wave (or distance between two successive ridges on the surface of the water), the orbits of the particles are nearly circular, and the motion at great depths is insensible.

The *period* of a wave is the time occupied by each particle in making one revolution, and is also the time occupied by a wave in travelling a distance equal to its length. Hence we have the following proportion:—

$$\frac{\text{mean speed of a particle}}{\text{speed of the waves}} = \frac{\text{circumference of particle's orbit}}{\text{length of a wave}}.$$

The *speed of the waves* depends principally on their length and on the depth of water, being greatest for long waves and deep water. When the depth of water is greater than the length of a wave the speed is not sensibly affected by the depth, and is almost exactly equal to the velocity acquired by a body in falling through *half of the radius of a circle whose circumference is the length of a wave.* In water that is very shallow, compared with the length of the waves, the velocity is nearly independent of the length, and is nearly equal to that acquired by a heavy body in falling through *half the depth of the water added to three-fourths of the height of a wave.*

Two or more different series of waves moving in the same, different, and contrary directions, with equal or unequal speeds, may traverse the same mass of water at the same time, and the motion of each particle of water will be the resultant of the respective motions which the several series of waves would have impressed upon it had they acted separately. This is called the *interference* of waves.

When a series of waves advances into water gradually becoming shallower, their *periods* remain unchanged, but their *speed,* and consequently their *length,* diminishes, and their slopes become steeper. The orbits of the particles of water become distorted, as

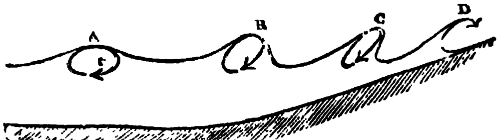


Fig. 294.

at B, C, D, fig. 294, in such a manner that the front of each wave gradually becomes steeper than the back; the crest, as it were, advancing faster than the trough. At length the front of the wave curls over beyond the vertical, its crest falls forward, and it *breaks* into surf on the beach.

As the energy of the motion of a given wave which advances

into shallowing water, or up a narrowing inlet, is successively communicated to smaller and smaller masses of water, there is a *tendency* to throw those masses into more and more violent agitation: that tendency may either take effect, or it may be counteracted, or more than counteracted, by the loss of energy which takes place through the production of eddies and surge at sudden changes of depth, and through friction on the bottom.

When waves roll straight against a vertical wall, as in fig. 295, they are reflected, and the particles of water for a certain distance in front of the wall have motions compounded of those due to the direct and to the reflected waves. The results are of the following kind:—The particles in contact with the wall, as at A, move up and down through a height equal to *double* the original height of the waves, and so also do those at half a wave length from the wall, as at C; the particles at a quarter of a wave length from the wall, as at B, move backwards and forwards horizontally, and intermediate particles oscillate in lines inclined at various angles.

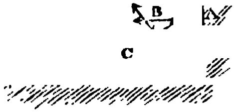


Fig. 295.

In order that a surface may reflect the waves, it is not essential that it should be exactly vertical; according to Mr. Scott Russell, it will do so even with a batter of 45°.

A vertical or steep surface which is wholly covered by the water reflects the wave-motion of those layers of water which lie below its level, and thus a sunken rock or breakwater, even though covered with water to a considerable depth, causes the sea to break over it, and so diminishes the energy of the advancing waves.

The *greatest length* of waves in the ocean is estimated at about 560 feet, which corresponds to a speed of about 53 feet per second, and a period of about 11 seconds. Their *greatest height* is given by Scoresby as about 43 feet, and this, with the period just stated, gives 12 feet per second as the velocity of revolution of the particles of water. (See p. 766.)

In smaller seas the waves are both lower and shorter, and less swift; and, according to Mr. Scott Russell, waves in an expanse of shallow water of nearly uniform depth never exceed in height the undisturbed depth of the water. But the concentration of energy upon small masses of water, which occurs on shelving coasts in the manner already stated, produces waves of heights greatly exceeding those which occur in water of uniform depth, as the following examples show.

Pressures of waves against a vertical surface, at Skerryvore as observed by Mr. Thomas Stevenson:—

	Summer average.	Winter average.	Storms
In lbs. per square foot, ...	611	2086	6083
In feet of water, ...	9·8	33	97

Greatest height of breakers on the south-west coast of Ireland, as observed by the Earl of Dunraven, 150 feet.

Recent investigations tend towards the conclusion, which is in accordance with observation, that every wave is more or less a "wave of translation," setting down each particle of water, or of matter suspended in water, a little in advance of where it picked that particle up, and thus by degrees producing that heaping up of water which gathers on a lee shore during a storm. This property of waves accounts for the facts, that although they tend to undermine and demolish steep cliffs, they heap up sand, gravel, shingle, or such materials as they are able to sweep along, upon every flat or sloping beach against which they directly roll; that they carry such materials into bays and estuaries; and that when they advance obliquely along the coast they make the materials of the beach travel along the coast in the same direction. (See p. 802.)

513. **Tides in General.**—The general motion of the tides consists in an alternate vertical rise and fall, and horizontal ebb and flow, occupying an average period of half a lunar day, or about 12·4 hours, and transmitted from place to place in the seas like a series of very long and swift waves, in which the extent of the horizontal motion is very much greater than that of the vertical motion. The extent of motion, both vertical and horizontal, undergoes variations between spring and neap, whose period is half a lunation, and other variations whose periods are a whole lunation and half-a-year. The propagation of the tide-waves is both retarded and deflected in gradually shallowing water, the crests of the waves having a tendency to become parallel to the line of coast which they are approaching.

Tides in narrow seas, and in the neighbourhood of land generally, are modified by the interference of different series of waves arriving by different routes, so as sometimes to present very complex phenomena. (See authorities, p. 766). In the following examples simple cases only are described.

514. **Tidal Waves in a Clear and Deep Channel** are analogous to ordinary waves, as represented in fig. 293, p. 753; but with the modification that, owing to the enormous length of the waves as compared with the depth of the sea, the extent of horizontal motion is nearly equal at all depths, and the extent of vertical motion in any layer is nearly in the simple proportion of its height above

the bottom. The orbit of each particle is a very long and flat ellipse.

Supposing such a channel as that here considered to have a beach of moderately steep slope at one side, the depth being elsewhere uniform, the particles near that beach move in ellipses situated in planes inclined so as to be nearly parallel to the beach, as represented in plan in figs. 296 and 297. In each of these figures the

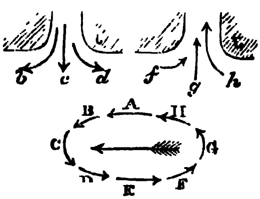


Fig. 296

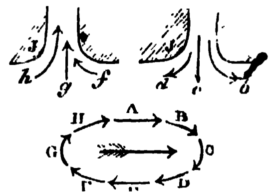


Fig. 297.

beach is supposed to be towards the top of the page; in fig. 296 it lies to the right hand of the direction of advance of the tide wave (represented by the feathered arrow); in fig. 297, to the left of that direction. The following are the motions of a particle at different times of the tide:—

Lunar Hours after High-water.	Time commonly called.	Current.	Reference to the Figures.
0	High-water,....................	Forward,........	A
1½	Quarter Ebb,....................	Forward and Seaward,......... .	B
3	Half Ebb,......................	Seaward,.........................	C
4½	Three-quarters Ebb,............	Backward and Seaward,..........	D
6	Low-water,.....................	Backward,........	E
7½	Quarter Flood,..................	Backward and Shoreward,.......	F
9	Half Flood,.....................	Shoreward,......................	G
10½	Three-quarters Flood,..	Forward and Shoreward,....	H
12	High-water,....................	Forward,.........................	A

515. The **Tide in a Short Inlet,** or in any bay gulf, or estuary of such dimensions and figure that high and low-water occur in all parts of it sensibly at the same instant, is somewhat analogous to a wave rising and falling against a steep wall (fig. 295, p. 755), or to the emptying and filling of a reservoir. Each particle of water moves alternately outwards and inwards during the fall and rise of the tide respectively; and the current is swifter and stronger when the depth of water is greater, that is, *during the second half of flood and the first half of ebb.*

Supposing that the entrance to such an inlet runs at right angles to the line of coast described in the preceding article, the combination of the tidal currents of the inlet with those of the offing, or sea outside, produces the results, as regards the currents at the

entrance, indicated by the arrows marked *b, c, d, f, g, h,* in figs. 296 and 297 (whose lengths denote the strength of the current), and explained in the following table,* in which ***outward*** and ***inward*** refer to the entrance of the inlet, and ***forwards*** and ***backwards*** to the directions of currents as compared with that of the flood-current along the coast:—

Lunar Hours after High-water.		Time commonly called		Reference to the Figures.	
First half of Ebb.	0	High-water,..........	0 (Slack-water),..................		
	$1\frac{1}{2}$	Quarter Ebb,.........	Outward, turning forward,.....		Strong.
	3	Half Ebb, ...	Outward,.........................		
Second half of Ebb.	$4\frac{1}{2}$	Three-quarters Ebb,	Outward, turning backward,...		Weak.
	6	Low-water,...........	0 (Slack-water),.		
First half of Flood.	$7\frac{1}{2}$	Quarter Flood,........	Backward, turning inward,....	*f*	Weak.
	9	Half Flood, ...	Inward,..........................	*g*	Strong.
Second half of Flood.	$10\frac{1}{2}$	Three-quarters Flood	Forward, turning inward,......	*h*	
	12	High-water,..........	0 (Slack-water),.................	—	

The letter J in each figure marks the *up-stream corner* of the entrance as regards the flood-current along the coast.

The *volume of water* which flows alternately in and out at the entrance of a short inlet is nearly equal to the space between the surfaces of high and low-water, as ascertained by levelling and tide-gauges. The *mean velocity* of the current through the entrance is nearly equal to that volume divided by the mean sectional area of the entrance, and by the time of rise or fall; and the *greatest velocity* is nearly equal to 1·57 × mean velocity. It is best to use such calculations only for the purpose of computing the probable effect of alterations. The velocities of actual currents should be found by observation.

516. The **Tides in Long Inlets** are compounded of a simple emptying and filling current like that in a short inlet, and a series of branch tidal waves, propagated up the channel from the waves of the offing. In *river-channels* the alternate currents due to the tides are combined with the downward current due to the flow of fresh water.

The tidal wave which is propagated up a long inlet or river-channel is analogous to those represented as advancing into shallow water in fig. 294, p. 754. It diminishes in length and increases in height until it reaches a limit where its further increase in height is stopped by friction. Its front becomes shorter and steeper, and its back longer and flatter; in other words, the rise of tide occupies a shorter time, and the fall a longer time, as the wave advances up the channel. When a high tidal wave advances into very shallow water, its front sometimes shortens and steepens, until at length it curls over, like the breaker D in fig. 294, and continues to advance

rolling and breaking into surf, followed by a very long flat back. The tidal wave is then called a "*bore.*" The back of the wave sometimes breaks up into two or three smaller waves, and then the fall of the tide is interrupted by short intervals of rise.

To estimate by calculation the velocity of the flood and ebb-currents at a given cross-section of a river-channel or other long inlet, two longitudinal sections of the surface of the water must be prepared from two sets of simultaneous tide-gauge observations, made at a series of stations along the channel and above that cross-section, *at the two instants of slack-water at the given cross-section* respectively. The volume contained between the two surfaces thus determined will be the volume of tidal water which runs in and out through the given cross-section; and this, being divided by the duration of flood and ebb respectively, and by the area, will give the probable mean velocities of the currents, which, being multiplied by 1·57, will give, approximately, the probable maximum velocities. The velocity due to the fresh-water stream, if any, is to be subtracted from the flood and added to the ebb. (See the remark at the end of the preceding article.)

The tidal waves in rivers are propagated up the declivity of the stream, which they often affect at points above the level of high water in the sea.

517. **Actions of Tides on Coasts and Channels.**—The flowing tide augments, and the ebbing tide diminishes, the speed and force of storm waves; and hence the observed fact, that the most powerful action of such waves on the coast occurs after half-flood, when the shoreward current is strong. The tidal currents sweep along with them silt or mud, sand, gravel, and other materials, according to the laws already stated with reference to river currents (Article 468, p. 708); hence the ebbing tide tends to scour and deepen inlets, and the flowing tide to silt them up. From what has been explained in the preceding article, it appears that in shallow water there is a tendency for the flowing tide to become more rapid, and therefore stronger in its action, than the ebbing tide, unless opposed by a sufficiently strong fresh-water current; and hence the prevailing tendency of the tides, like that of the waves, is to choke and fill up estuaries, river-channels, and other inlets, especially such as are already shallow.

A strong fresh-water current may maintain a deep channel against this action of the sea, so far as it is limited in breadth; but where that current escapes into the open sea, and is either enfeebled by spreading laterally, or has its action on the bottom prevented by floating on the salt water, a *bar* is formed by the action of the waves and tides.

One of the chief objects of harbour engineering is so to manage

and modify the action of the tidal currents that the ebb shall become stronger than the flood, and shall scour deep channels and remove bars. (See p. 766.)

SECTION II.—*Of Sea Defences.**

518. **Groins**, running out at right angles to the coast, are constructed in the same manner with groins for river-banks, but more strongly. (Article 469, p. 711.) They not only interrupt the *travelling* of the materials of the beach along the shore under the influence of oblique waves and of the flowing tide, but they also cause a permanent deposit of such materials, and, if gradually extended seaward in shallow water, produce a gain of ground from the sea. After the spaces between the groins have been filled up, the travelling of shingle goes on past their ends as before.

Groins are amongst the most efficient means of protecting dykes, cliffs, and sea-walls, against the undermining action of the sea.

519. An **Earthen Dyke** has usually a long flat slope towards the sea, its inclination ranging from that of 3 to 1 to that of 12 to 1. The top is level, and usually has a roadway upon it: its average usual height above high-water-mark of spring tides, is about 6 feet; it should, if possible, be above the reach of the waves. The back slope has an inclination ranging from that of $1\frac{1}{2}$ to 1 to that of 3 to 1. Behind the dyke is a back drain, or ditch, for the drainage of the land, constructed on the same principles with the back drains mentioned in Article 483, p. 727, and Article 484, p. 728.

In the heart of the dyke is a rectangular wall of fascines, constructed like the fascine-work of a river-bank. (Article 469, p. 710.) The fascines may be made of willow twigs or of reeds. The seaward slope is faced with fascines. If the top is above the reach of the waves the back slope may be turfed; if waves sometimes break over it, the top and back require stone pitching.

520. **Stone Bulwarks** withstand the waves best when either very flat or very steep. They are of two principal kinds—those with a long slope, on which the waves break, as in fig. 294, p. 754, and those with a steep face, which reflect the waves as in fig. 295, p. 755.

I. *Long-sloping Bulwarks* have an inclination which ranges from 3 to 1 to 7 to 1. They are made internally of earth and gravel, or of loose stones, according to the situation, and are faced with blocks, each of which should be able to withstand independently the lifting action of the waves. As to this, see Article 412, p. 618. The foot or "toe" of the slope may be slightly turned up, like that of a weir, to prevent the undermining action of the returning current, or "undertow" from the breakers. (See Article 472, p. 713.)

* See also Appendix.

To prevent breakers or spray from gliding up to the top of the slope, and dashing over the summit of the bulwark, the top of the slope is sometimes curved upwards, so as to present a concave face to the waves; but this is sometimes liable to be knocked down by the shocks which it receives; and in that case it is best to carry up the slope in one plane, with a level *berm* or bench at the top of it, paved with large blocks, and on that berm to erect a strong parapet, set so far back that its cope is below the plane of the slope. A series of level berms, alternating with flat slopes of the same length with the berms, or thereabouts, are very effective in breaking the waves and exhausting their energy; the blocks at the edges of the berms must be larger than the rest.

The largest blocks in the facing of the slope should be at and near half-tide level, because the waves are largest at half-flood.

When a sloping bulwark stands in deep water, the part below low-water-mark may have a steeper slope than that above, as being less violently acted upon by the waves: for example, from 1 to 1 to 3 to 1 below, and from 4 to 1 to 7 to 1 above. The waves will partially break and lose their energy in passing over the place where the inclination changes.

II. A *Steep-faced Bulwark or Sea-Wall* should be proportioned like a reservoir wall. (See Article 465, p. 707.) As to the manner in which it reflects the waves, see Article 512, p. 755. Its cope should either rise above the crests of the highest waves, augmented as they are in height by the reflection, or, should that be impracticable, that cope should be made of stones, each large enough to resist being lifted by the pressure due to the greatest height of a wave above its bed, and dowelled to the adjoining cope-stones. The front edge of the cope should not project beyond the face of the wall, lest the waves overturn it. The remainder of the wall may have a hammer-dressed ashlar or a block-in-course face, backed with coursed rubble or with strong concrete, the whole built in strong hydraulic mortar, and the outer edges of the joints *laid* in cement. (Article 248, p. 389.) The chief danger to the face of such a wall is that air and water should penetrate the joints, and, by their pressure and elasticity, cause stones to jump out after receiving the blow of a wave.

The undermining action of the waves on the ground at the foot of a steep wall is very severe, and should be resisted by a flat stone pitching (which should have no bond or connection with the wall), and by a series of groins. The undermining action may be somewhat moderated by forming the face of the wall into steps, so as to interrupt the vertical descent of the water.

There are good grounds for believing it to be advantageous to build sea-walls in courses of stones which stand nearly on edge,

instead of lying horizontal, in order that each stone may always be loaded with the whole weight of those directly above it.

When there is an earthen embankment behind a sea wall, it should have a retaining wall at the landward side also, to prevent the earth from being washed away by water which may collect on the top.

III. *Combined Wall.*—As the expense of erecting a steep or vertical wall in deep water is very great, it is sometimes combined in such situations with a long slope, in the following manner:—From the bottom up to near low-water-mark extends a slope of 2 to 1 or 3 to 1, terminating in a long level or nearly level *berm* or "*foreshore;*" and on that berm, as on a beach in shallow water, is built a steep wall, at a distance back from the edge of the slope equal to twice or thrice the length of the slope.

521. A **Breakwater,** being placed so as to defend a harbour or roadstead from the waves, differs from a bulwark by having sea at both sides of it. The site of a breakwater should be so chosen as to present a barrier to the waves of the prevailing storms, and especially to those which come along with the flood-current. It may be isolated, and in the midst of the entrance of a bay, as at Plymouth and Cherbourg, or it may run out from the shore into deep water. In the latter case, the best position for the junction of a single breakwater with the land is in general at the *up-stream corner* of the entrance to the inlet or harbour (see Article 515, p. 758), for in that position it opposes the strongest flood-current, and does not interfere with the strongest ebb-current. The principles of the construction of the front of a breakwater are the same with those described in the preceding article with reference to bulwarks in deep water. The back of a vertical-fronted breakwater is usually vertical also; that of a sloping or combined breakwater, if intended to be used as a quay, is vertical; in other cases it differs from the front only in having a steeper slope (from 1 to 1 to $1\frac{1}{2}$ to 1) and being faced with smaller blocks. As to embanking and building under water, see Article 412, p. 617. When a stage supported on screw piles is used to tip the stones from, those piles remain imbedded in the breakwater. Their diameter should be about $\frac{1}{20}$th of their height, so that, in very deep water, they may require to be built of several balks of timber hooped together, as at Portland.

Fig. 298 is a section of the Cherbourg breakwater, which combines the long slope and vertical face. The base A F is about 300 feet; the slope A B is $2\frac{1}{2}$ to 1; B C is $5\frac{1}{2}$ to 1; E F, 1 to 1; C D is a nearly level platform, on which stands the wall G, 36 feet thick at its base. Ordinary spring tides rise 19 feet, the depth at low-water being 40 feet.

Fig. 299, a section of the Plymouth breakwater, illustrates the

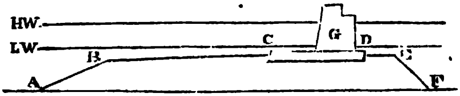


Fig. 298.

principle of alternate slopes and berms. A B is 3 to 1, B C level, C D 5 to 1, D E level, E F 1½ to 1.

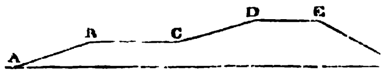


Fig. 299.

(As to breakwaters, and sea defences generally, may be consulted the works of Smeaton and Telford, Sir John Rennie's works *On the Plymouth Breakwater* and *On Harbours*, the *Proceedings of the Institution of Civil Engineers* since the commencement, and Mr. Burnell's *Treatise on Marine Engineering*.) (See also pp. 766, 793.)

522. **Reclaiming Land.**—The process of reclaiming or gaining land from the sea is to be undertaken with great caution, especially in river-channels and estuaries, lest it should diminish the tidal scour, and so cause the silting up of channels and harbours; and, in particular, care should be taken that the space for tidal water which is to be lost through the reclaiming of the land, is exactly made up for by deepening or otherwise improving other parts of the estuary or channel. In every instance in which that precaution has been neglected, the damage, and in some cases the ruin, of the harbour has followed. (See *Reports of the Tidal Harbours Commission.*)

The first operation in reclaiming land is usually to raise its level as much as possible by *warping*, or deposition of sediment from the tidal water; with a view to which the land to be reclaimed is intersected by a network of transverse wattled groins, and of longitudinal dykes of the same construction.

The ground having been raised as far as practicable by warping, is enclosed with sea-dykes, and drained in the manner described in Article 484, p. 727.

SECTION III.—*Of Tidal Channels and Harbours.*

523. The **Improvement of Tidal Rivers and Estuaries** depends mainly on the strengthening of the ebbing current, as stated in Article 517, p. 760. With that view, the measures to be adopted

are nearly the same with those already described under the head of Improvements of River-Channels, Article 470, p. 711, with the addition that the space which at each tide is filled and emptied is to be kept as large as possible. For the purpose of concentrating the latter portions of the ebbing current upon the deep-water-channel, *training-dykes* may be required. That these may not diminish the quantity of scouring-water, they should rise but little, if at all, above low-water-mark of ordinary spring-tides, their position being marked by means of rows of beacons.

Should bulwarks or quays be erected, they should either be so placed that the area which they cut off by contracting wide places may be compensated for by widening narrow places, or that the space which they cut off may be compensated for by deepening that part of the space in front of them which is above low-water-mark.

The most important effect of making a deep, direct, and regular channel for a tidal river consists in the increase in the extent of rise and fall of the tide, and the diminution of that steepening action of a shallow channel on the front of the tide-wave which has been described in Article 516, p. 758.

In order to increase the depth over a *bar*, piers or breakwaters must be carried out so as to concentrate the current over it, and it is best, if possible, to make the space between those piers *widen inwards*, in order both to hold scourage-water and to serve as a "wave-trap," or space for storm-waves which roll in at the entrance to spread and expend themselves in. When there is only one pier, it should run from the *up-stream* corner of the entrance, for the reason explained in Article 521, p. 762, observing that in deciding which is the *up-stream* corner, regard must be had to the flood-current *along the shore*, in case, through the action of headlands, its direction should be different from that of the flood-current in the open sea.

The bar may thus be swept into deeper water, although it is in general impossible to remove it altogether.

524. A **Scouring-Basin** is a reservoir by means of which the tidal water is stored up to a certain level, and let out through sluices, in a rapid stream, for a few minutes at low-water, to scour a channel and its bar. The outlets of the basin should face as nearly as possible directly along the channel to be scoured; they should be distributed throughout its whole cross-section, that they may produce an uniform steady current in it like a river, and may not concentrate their action on a few spots. To carry away gravel and large shingle, the scouring stream should flow at 4 or 5 feet per second, and the dimensions of the outlets should be regulated accordingly. One of the best examples of such an arrangement is

at the south entrance of the harbour of Sunderland, described by the engineer, Mr. Murray, in the *Proceedings of the Institution of Civil Engineers* for 1856. The current is let out for 15 minutes at low-water; it runs at about 5 feet per second, and is sensible in the sea 2,000 yards off, although it is confined by piers for 350 yards only.

525. **Quays** of masonry are to be regarded as a class of retaining walls, the stability of which has been treated of in Articles 265 to 269, pp. 401 to 408, and their construction in Articles 271, 272, pp. 409 to 411. Their ordinary thickness at the base is from $\frac{1}{3}$ to $\frac{1}{4}$ of their height. When founded on piles, the timber-work should be always immersed. (See Part II., Chapter VI., Section II., p. 601.). The face of a stone quay is usually protected against being damaged by vessels by means of a network of upright *fender-piles* and horizontal *fender-wales*.

As to timber and iron quays, see Article 469, p. 710, and the other articles there referred to.

The inner side of a breakwater may form a quay, as already mentioned.

526. **Piers** of masonry running out into the sea are to be regarded as upright breakwaters combined with quays, and require here no additional explanation. Those of timber and iron are best formed of a skeleton framework, supported by screw-piles. A timber skeleton-pier is often combined with a loose stone breakwater, in which the lower parts of the posts are imbedded.

527. **Basins and Docks**.—A *deep-water-basin* is a reservoir surrounded by quay-walls, in which the water is retained when the tide falls below a certain level (usually somewhat above half-tide), by a pair of lock-gates opening inwards, of sufficient size and strength. Should the entrance be exposed to waves, a pair of *sea-gates*, or gates opening outwards, are also required, to be closed during storms. A deep-water-basin may also be used as a scouring-basin. (Article 524, p. 764.)

A *dock* differs from a basin in having a *lock* at its entrance, through which ships can pass in all states of the tide. (As to locks, see Article 506, p. 746.) A harbour-lock, like a river-lock, has no lift-wall. In order that vessels may pass easily in and out, the entrances of docks from a river-channel should slant *up-stream as regards the ebb-current.*

One of the best forms of gate for basins and docks is a *caisson-gate*, being a water-tight vessel of plate-iron, which can be floated to or from its seat in the masonry of the entrance, being placed in a recess when open. When closed it is sunk by loading it with water, which is run into a tank on the top of the caisson. In order to open it, it is floated by emptying that tank.

It is often convenient, when practicable, to conduct a supply of fresh water into basins or docks, care being taken that such supply is pure.

528. **Lighthouses.**—The principles which regulate the placing and illuminating of lighthouses form a subject which can be fully considered in a special treatise only, such as that by Mr. Thomas Stevenson. When a lighthouse is exposed to the waves, it may be either a round tower of masonry, built of hewn stones, dove-tailed, tabled, and dowelled to each other, as described in Article 412, p. 618, solid up to the level of high-water of spring tides, and as much higher as ordinary waves rise, and high enough in all to keep the lantern clear of the highest breaking and reflected storm-waves, with an overhanging curved cornice to throw their crests back; or it may consist of a skeleton frame of screw-piles and diagonal bracing, supporting a timber or iron house and platform; and in this case the platform needs only to be high enough to clear the tops of the natural unreflected waves. On the subject of the strength and stability of frames supported on screw-piles, see Article 403, p. 605. In designing the frame of a lighthouse to be supported on them, regard must be had to the pressure of the wind, whose greatest recorded intensity, in Britain, is 55 lbs. per square foot of a flat surface, and about one-half of that intensity per square foot of the plane projection of a cylindrical surface.

Additional Authorities on Harbour and Sea Works.

Minard—*Ouvrages Hydrauliques des Ports de Mer.* Bremner *On Harbours* (Wick, 1845). Thomas Stevenson *On the Design and Construction of Harbours*

Lighthouses.—Smeaton's *Account of the Eddystone Lighthouse.* Robert Stevenson *On the Bell Rock Lighthouse.* Alan Stevenson *On the Skerryvore Lighthouse.* Alan Stevenson, *Rudimentary Treatise on Lighthouses* Thomas Stevenson *On Lighthouse Illumination.* Mitchell's "Account of Lighthouses on Screw Piles," in the *Proceedings of the Institution of Civil Engineers* for 1848.

Waves.—J. Scott Russell; *Reports of the British Association* for 1844. G. G. Stokes; *Cambridge Transactions*, 1842, 1850. Earnshaw; *ib.*, 1845. W Froude; *Transactions of the Institution of Naval Architects*, 1862. Rankine; *Philosophical Transactions*, 1863 Watts, Rankine, Napier, and Barnes, *On Shipbuilding*, 1861. Cialdi: *Sul Moto ondoso del Mare*, 1866. Caligny: *Liouville's Journal*, June and July, 1866.

Harbours.—James Deas, *The River Clyde;* L. F. Vernon Harcourt *Harbours and Docks;* Stevenson, *Design and Construction of Harbours.*

Addendum to Article 512, p. 755.—Height of Waves.—The height of the waves depends on what is called the "Fetch;" that is, the distance from the weather shore, where their formation commences. According to Mr. Thomas Stevenson, the following formula is nearly correct during heavy gales, when the fetch is not less than about six nautical miles; height in feet $= 1{\cdot}5 \times \sqrt{\text{(fetch in nautical miles)}}$.

Addendum to Article 517, p. 759.—Scouring Action of Tide.—According to Mr. Thomas Stevenson, the sectional area of many estuaries at *low water* bears a nearly constant proportion to the volume of water which runs in and out at each tide, being from $7\frac{1}{2}$ to 10 square feet of area for each 1,000,000 cubic feet of tidal water.

APPENDIX.

I.

Table of the Resistance of Materials to Stretching and Tearing by a Direct Pull, *in pounds avoirdupois per square inch.*

Materials.	Tenacity, or Resistance to Tearing.	Modulus of Elasticity, or Resistance to Stretching.
Stones, Natural and Artificial:		
Brick, Cement,	280 to 300	
Glass,	9,400	8,000,000
Slate,	9,600 to 12,800	13,000,000 to 16,000,000
Mortar, ordinary,	50	
Metals:		
Brass, cast,	18,000	9,170,000
" wire,	49,000	14,230,000
Bronze or Gun Metal (Copper 8, Tin 1),	36,000	9,900,000
Copper, cast,	19,000	
" sheet,	30,000	
" bolts,	36,000	
" wire,	60,000	17,000,000
Iron, cast, various qualities,	13,400 to 29,000	14,000,000 to 22,900,000
" average,	16,500	17,000,000
Iron, wrought, plates,	51,000	
" joints, double rivetted,	35 700	
" " single rivetted,	28,600	
" bars and bolts,	60,000 to 70,000	29,000,000
" hoop, best-best,	64,000	
" wire,	70,000 to 100,000	25,300,000
" wire-ropes,	90,000	15,000,000
Lead, sheet,	3,300	720,000
Steel plates, average, bridge work,	66,000	
" " boiler "	53,000 to 70,000	
Cast steel,	70,000	
Tin, cast,	4,600	
Zinc,	7,000 to 8,000	

Materials.	Tenacity, or Resistance to Tearing.	Modulus of Elasticity, or Resistance to Stretching.
Timber and other Organic Fibre:		
Acacia, false. See "Locust."		
Ash (*Fraxinus excelsior*),..........	17,000	1,600,000
Bamboo (*Bambusa arundinacea*),	6,300	
Beech (*Fagus sylvatica*), ...	11,500	1,350,000
Birch (*Betula alba*),................	15,000	1,645,000
Box (*Buxus sempervirens*),........	20,000	
Cedar of Lebanon (*Cedrus Libani*),	11,400	486,000
Chestnut (*Castanea Vesca*),.......	10,000 to 13,000	1,140,000
Elm (*Ulmus campestris*),..........	14,000	700,000 to 1,340,000
Fir: Red Pine (*Pinus sylvestris*),	12,000 to 14,000	1,460,000 to 1,900,000
„ Spruce (*Abies excelsa*),.......	12,400	1,400,000 to 1,800,000
„ Larch (*Larix Europæa*),.....	9,000 to 10,000	900,000 to 1,360,000
Flaxen Yarn,..................about	25,000	
Hazel (*Corylus Avellana*),.........	18,000	
Hempen Ropes,..........from 12,000 to	16,000	
Hide, Ox, undressed,...	6,300	
Hornbeam (*Carpinus Betulus*),...	20,000	
Lancewood (*Guatteria virgata*),...	23,400	
Leather, Ox,........................	4,200	24,300
Lignum-Vitæ (*Guaiacum officinale*),............................	11,800	
Locust (*Robinia Pseudo-Acacia*),	16,000	
Mahogany (*Swietenia Mahagoni*),	8,000 to 21,000	1,255,000
Maple (*Acer campestris*), ...	10,600	
Oak, European (*Quercus sessiliflora* and *Quercus pedunculata*),	10,000 to 19,800	1,200,000 to 1,750,000
„ American Red (*Quercus rubra*),............................	10,250	2,150,000
Silk Fibre,............................	52,000	1,300,000
Sycamore (*Acer Pseudo-Platanus*),	13,000	1,040,000
Teak, Indian (*Tectona grandis*),	15,000	2,400,000
„ African, (?)..................	21,000	2,300,000
Whalebone,..........................	7,700	
Yew (*Taxus baccata*),..	8,000	

II.

Table of the Resistance of Materials to Shearing and Distortion, *in pounds avoirdupois per square inch.*

Materials.	Resistance to Shearing.	Transverse Elasticity, or Resistance to Distortion.
Metals:		
Brass, wire-drawn,		5,330,000
Copper,		6,200,000
Iron, cast,	27,700	2,850,000
„ wrought,	50,000	8,500,000 to 10,000,000
Timber:		
Fir: Red Pine,	500 to 800	62,000 to 116,000
„ Spruce,	600	......
„ Larch,	970 to 1,700	......
Oak,	2,300	82,000
Ash and Elm,	1,400	76,000

III.

Table of the Resistance of Materials to Crushing by a Direct Thrust, *in pounds avoirdupois per square inch.*

Materials.	Resistance to Crushing.
Stones, Natural and Artificial: (see also page 361).	
Brick, weak red,	550 to 800
„ strong red,	1,100
„ fire,	1,700
Chalk,	330
Granite,	5,500 to 11,000
Limestone, marble,	5,500
„ granular,	4,000 to 4,500
Sandstone, strong,	5,500
„ ordinary,	3,300 to 4,400
„ weak,	2,200
Rubble masonry, about four-tenths of cut stone.	
Metals:	
Brass, cast,	10,300
Iron, cast, various qualities,	80,000 to 145,000
„ „ average,	112,000
„ wrought,	about 36,000 to 40,000

MATERIALS.	Resistance to Crushing.
TIMBER,* Dry, crushed along the grain:	
Ash,	9,000
Beech,	9,360
Birch,	6,400
Blue-Gum (*Eucalyptus Globulus*),	8,800
Box,	10,300
Bullet-tree (*Achras Sideroxylon*),	14,000
Cabacalli,	9,900
Cedar of Lebanon,	5,860
Ebony, West Indian (*Brya Ebenus*),	19,000
Elm,	10,300
Fir: Red Pine,	5,375 to 6,200
„ American Yellow Pine (*Pinus variabilis*),	5,400
„ Larch,	5,570
Hornbeam,	7,300
Lignum-Vitæ,	9,900
Mahogany,	8,200
Mora (*Mora excelsa*),	9,900
Oak, British,	10,000
„ Dantzic,	7,700
„ American Red,	6,000
Teak, Indian,	12,000
Water-Gum (*Tristania nerifolia*),	11,000

IV

TABLE OF THE RESISTANCE OF MATERIALS TO BREAKING ACROSS, *in pounds avoirdupois per square inch.*

MATERIALS.	Resistance to Breaking, or Modulus of Rupture.†
STONES:	
Sandstone,	1,100 to 2,360
Slate,	5,000

* The resistances stated are for *dry* timber. Green timber is much weaker, having sometimes only half the strength of dry timber against crushing.

† The modulus of rupture is eighteen times the load which is required to break a bar of one inch square, supported at two points one foot apart, and loaded in the middle between the points of support.

MATERIALS.	Resistance to Breaking, or Modulus of Rupture.
METALS:	
Iron, cast, open-work beams, average,	17,000
„ „ solid rectangular bars, var. qualities,	33,000 to 43,500
„ wrought, plate beams,	42,000
Steel (varies with quality, see p. 767; also Appendix).	
TIMBER:	
Ash,	12,000 to 14,000
Beech,	9,000 to 12,000
Birch,	11,700
Blue-Gum,	16,000 to 20,000
Bullet-tree,	15,900 to 22,000
Cabacalli,	15,000 to 16,000
Cedar of Lebanon,	7,400
Chestnut,	10,660
Cowrie (*Dammara australis*),	11,000
Ebony, West Indian,	27,000
Elm,	6,000 to 9,700
Fir: Red Pine,	7,100 to 9,540
„ Spruce,	9,900 to 12,300
„ Larch,	5,000 to 10,000
Greenheart (*Nectandra Rodiæi*),	16,500 to 27,500
Lancewood,	17,350
Lignum-Vitæ,	12,000
Locust,	11,200
Mahogany, Honduras,	11,500
„ Spanish,	7,600
Mora,	22,000
Oak, British and Russian,	10,000 to 13,600
„ Dantzic,	8,700
„ American Red,	10,600
Poon,	13,300
Saul,	16,300 to 20,700
Sycamore,	9,600
Teak, Indian,	12,000 to 19,000
„ African,	14,980
Tonka (*Dipteryx odorata*),	22,000
Water-Gum,	17,460
Willow (*Salix*, various species),	6,600

(See also pp. 790, 793–795, 798–804.)

V.—Comparative Table of French and British Measures.

	No.	Log.	Log.	No.	
Grains in a gramme,..........	15·43235	1·188432	$\bar{2}$·811568	0·064799	Gramme in a grain.
Pounds avoird. in a kilogramme,	2·20462	0·343334	$\bar{1}$·656666	0·453593	Kilog. in a lb. avoirdupois.
Ton in a tonne,..................	0·984206	$\bar{1}$·993086	0·006914	1·01605	Tonnes in a ton.
Feet in a mètre, ...	3·2808693	0·515989	$\bar{1}$·484611	0·3047972 1	Mètres in a foot.
Inch in a millimètre,............	0·03937043	$\bar{2}$·595170	1·404830	25·39977	Millimètres in an inch.
Mile in a kilomètre,.............	0·621377	$\bar{1}$·793355	0·206645	1·60933	Kilomètres in a mile.
Square feet in a square mètre,..	10·7641	1·031978	$\bar{2}$·968022	0·0929013	Square mètre in a square foot.
Square inch in a square millimètre,........................	0·00155003	$\bar{3}$·190340	2·809660	645·148	Square millim. in a square inch.
Cubic feet in a cubic mètre,.....	35·3156	1·547967	$\bar{2}$·452033	0·0283161	Cubic mètre in a cubic foot.
Foot-pounds in a kilogrammètre,	7·23308	0·859323	$\bar{1}$·140677	0·138254	Kilogrammètre in a foot-pound.
Pounds-to-the-foot in a kilogramme-to-the-mètre,......	0·671963	$\bar{1}$·827345	0·172655	1·48818	Kilogrammes-to-the-mètre in a pound-to-the-foot.
Pounds-to-the-square-foot in a kilogramme-to-the-square-mètre,........................	0·204813	$\bar{1}$·311356	0·688644	4·88252	Kilogrammes-to-the-square-mètre in a pound-to-the-square-foot.
Pounds-to-the-square-inch in a kilog.-to-the-square-millimètre,......................	1422·31	3·152994	$\bar{4}$·847006	0·000703083	Kilog.-to-the-square-millimètre in a pound-to-the-square-inch.
Pounds-to-the-cubic-foot in a kilogramme-to-the-cubic-mètre,......................	0·062426	$\bar{2}$·795367	1·204633	16·019	Kilogrammes-to-the-cubic-mètre in a pound-to-the-cubic-foot.
Fahrenheit-degrees in a centigrade-degree,................	1·8	0·255273	$\bar{1}$·744727	0·5555̇5	Centigrade-degree in a Fahrenheit-degree.
British units of heat in a French unit,................	3·96832	0·598607	$\bar{1}$·401393	0·251996	French units of heat in a British unit.

VI.

Table of Specific Gravities of Materials.

	Weight of a cubic foot in lb. avoirdupois.
Gases, at 32° Fahr., and under the pressure of one atmosphere, of 2116·3 lb. on the square foot:	
Air	0·080728
Carbonic Acid	0·12344
Hydrogen	0·005592
Oxygen	0·089256
Nitrogen	0·078596
Steam (ideal)	0·05022
Æther vapour (ideal)	0·2093
Bisulphuret-of-carbon vapour (ideal)	0·2137
Olefiant gas	0·0795

	Weight of a cubic foot in lb. avoirdupois.	Specific gravity, pure water = 1.
Liquids at 32° Fahr. (except Water, which is taken at 39°·1 Fahr.):		
Water, pure, at 39°·1	62·425	1·000
„ sea, ordinary	64·05	1·026
Alcohol, pure	49·38	0·791
„ proof spirit	57·18	0·916
Æther	44·70	0·716
Mercury	848·75	13·596
Naphtha	52·94	0·848
Oil, linseed	58·68	0·940
„ olive	57·12	0·915
„ whale	57·62	0·923
„ of turpentine	54·31	0·870
Petroleum	54·81	0·878
Solid Mineral Substances, non-metallic:		
Basalt	187·3	3·00
Brick	125 to 135	2 to 2·167
Brickwork	112	1·8
Chalk	117 to 174	1·87 to 2·78
Clay	120	1·92
Coal, anthracite	100	1·602
„ bituminous	77·4 to 89·9	1·24 to 1·44
Coke	62·43 to 103·6	1·00 to 1·66
Felspar	162·3	2·6
Flint	164·2	2·63

	Weight of a cubic foot in lb. avoirdupois.	Specific gravity, pure water = 1
Solid Mineral Substances—*continued.*		
Glass, crown, average,	156	2·5
„ flint, „	187	3·0
„ green, „	169	2·7
„ plate, „	169	2·7
Granite,	164 to 172	2·63 to 2·76
Gypsum,	143·6	2·3
Limestone (including marble),	169 to 175	2·7 to 2·8
„ magnesian,	178	2·86
Marl,	100 to 119	1·6 to 1·9
Masonry,	116 to 144	1·85 to 2·3
Mortar,	109	1·75
Mud,	102	1·63
Quartz,	165	2·65
Sand (damp),	118	1·9
„ (dry),	88·6	1·42
Sandstone, average,	144	2·3
„ various kinds,	130 to 157	2·08 to 2·52
Shale,	162	2·6
Slate,	175 to 181	2·8 to 2·9
Trap,	170	2·72
Metals, solid:		
Brass, cast,	487 to 524 4	7·8 to 8·4
„ wire,	533	8·54
Bronze,	524	8·4
Copper, cast,	537	8·6
„ sheet,	549	8·8
„ hammered,	556	8·9
Gold,	1186 to 1224	19 to 19·6
Iron, cast, various,	434 to 456	6·95 to 7·3
„ average,	444	7·11
Iron, wrought, various,	474 to 487	7·6 to 7·8
„ average,	480	7·69
Lead,	712	11·4
Platinum,	1311 to 1373	21 to 22
Silver,	655	10·5
Steel,	487 to 493	7·8 to 7·9
Tin,	456 to 468	7·3 to 7·5
Zinc,	424 to 449	6·8 to 7·2

TIMBER :*	Weight of a cubic foot in lb. avoirdupois.	Specific gravity, pure water = 1
Ash,	47	0·753
Bamboo,	25	0·4
Beech,	43	0·69
Birch,	44·4	0·711
Blue-Gum,	52·5	0·843
Box,	60	0·96
Bullet-tree,	65·3	1·046
Cabacalli,	56·2	0·9
Cedar of Lebanon,	30·4	0·486
Chestnut,	33·4	0·535
Cowrie,	36·2	0·579
Ebony, West Indian,	74·5	1·193
Elm,	34	0·544
Fir: Red Pine,	30 to 44	0·48 to 0·7
„ Spruce,	30 to 44	0·48 to 0·7
„ American Yellow Pine,	29	0·46
„ Larch,	31 to 35	0·5 to 0·56
Greenheart,	62·5	1·001
Hawthorn,	57	0·91
Hazel,	54	0·86
Holly,	47	0·76
Hornbeam,	47	0·76
Laburnum,	57	0·92
Lancewood,	42 to 63	0·675 to 1·01
Larch. See "Fir."		
Lignum-Vitæ,	41 to 83	0·65 to 1·33
Locust,	44	0·71
Mahogany, Honduras,	35	0·56
„ Spanish,	53	0·85
Maple,	49	0·79
Mora,	57	0·92
Oak, European,	43 to 62	0·69 to 0·99
„ American Red,	54	0·87
Poon,	36	0·58
Saul,	60	0·96
Sycamore,	37	0·59
Teak, Indian,	41 to 55	0·66 to 0·88
„ African,	61	0·98
Tonka,	62 to 66	0·99 to 1·06
Water-Gum,	62·5	1·001
Willow,	25	0·4
Yew,	50	0·8

* The Timber in every case is supposed to be dry.

TABLE OF SQUARES AND FIFTH POWERS.

	Square.	Fifth Power.		Square.	Fifth Power.
10	1 00	1 00000	55	30 25	5032 84375
11	1 21	1 61051	56	31 36	5507 31776
12	1 44	2 48832	57	32 49	6016 92057
13	1 69	3 71293	58	33 64	6563 56768
14	1 96	5 37824	59	34 81	7149 24299
15	2 25	7 59375	60	36 00	7776 00000
16	2 56	10 48576	61	37 21	8445 96301
17	2 89	14 19857	62	38 44	9161 32832
18	3 24	18 89568	63	39 69	9924 36543
19	3 61	24 76099	64	40 96	10737 41824
20	4 00	32 00000	65	42 25	11602 90625
21	4 41	40 84101	66	43 56	12523 32576
22	4 84	51 53632	67	44 89	13501 25107
23	5 29	64 36343	68	46 24	14539 33568
24	5 76	79 62624	69	47 61	15640 31349
25	6 25	97 65625	70	49 00	16807 00000
26	6 76	118 81376	71	50 41	18042 29351
27	7 29	143 48907	72	51 84	19349 17632
28	7 84	172 10368	73	53 29	20730 71593
29	8 41	205 11149	74	54 76	22190 06624
30	9 00	243 00000	75	56 25	23730 46875
31	9 61	286 29151	76	57 76	25355 25376
32	10 24	335 54432	77	59 29	27067 84157
33	10 89	391 35393	78	60 84	28871 74368
34	11 56	454 35424	79	62 41	30770 56399
35	12 25	525 21875	80	64 00	32768 00000
36	12 96	604 66176	81	65 61	34867 84401
37	13 69	693 43957	82	67 24	37073 98439
38	14 44	792 35168	83	68 89	39390 40643
39	15 21	902 24199	84	70 56	41821 19424
40	16 00	1024 00000	85	72 25	44370 53125
41	16 81	1158 56201	86	73 96	47042 70176
42	17 64	1306 91232	87	75 69	49842 09207
43	18 49	1470 08443	88	77 44	52773 19168
44	19 36	1649 16224	89	79 21	55840 59449
45	20 25	1845 28125	90	81 00	59049 00000
46	21 16	2059 62976	91	82 81	62403 21451
47	22 09	2293 45007	92	84 64	65908 15232
48	23 04	2548 03968	93	86 49	69568 83693
49	24 01	2824 75249	94	88 36	73390 40224
50	25 00	3125 00000	95	90 25	77378 09375
51	26 01	3450 25251	96	92 16	81537 26976
52	27 04	3802 04032	97	94 09	85873 40257
53	28 09	4181 95493	98	96 04	90392 07968
54	29 16	4591 65024	99	98 01	95099 00499

* **Towers and Chimneys** are exposed to the lateral pressure of the wind, which, without sensible error in practice, may be assumed to be horizontal, and of uniform intensity at all heights above the ground.

The surface exposed to the pressure of the wind by such structures is usually either flat, or cylindrical, or conical, and differing very little from the cylindrical form. Octagonal chimneys, which are occasionally erected, may be treated as sensibly circular in plan. The inclination of the surface of a tower or chimney to the vertical is seldom sufficient to be worth taking into account in determining the pressure of the wind against it.

The greatest intensity of the pressure of the wind against a flat surface directly opposed to it hitherto observed in Britain, has been 55 lbs. per square foot; and this result, obtained by observations with anemometers, has been verified by the effects of certain violent storms in destroying factory chimneys and other structures.

In any other climate, before designing a structure intended to resist the lateral pressure of wind, the greatest intensity of that pressure should be ascertained, either by direct experiment, or by observation of the effects of the wind on previous structures.

The total pressure of the wind against the side of a cylinder is about one-half of the total pressure against a diametral plane of that cylinder.

Let fig. 98 represent a chimney, square or circular, and let it be required to determine the conditions of stability of a given bed-joint D E.

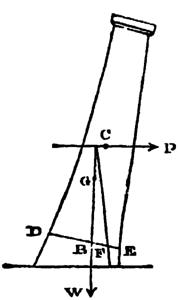


Fig. 98.

Let S denote the area of a diametral vertical section of the part of the chimney above the given joint, and p the greatest intensity of pressure of the wind against a flat surface. Then the total pressure of the wind against the chimney will be sensibly

$$\left.\begin{array}{ll} P = p\,S & \text{for a square chimney;} \\ P = p\,\dfrac{S}{2} & \text{for a round chimney;} \end{array}\right\} \ldots (1.)$$

and its resultant may, without appreciable error, be assumed to act in a horizontal line through the *centre of gravity of the vertical diametral section*, C. Let H denote the height of that centre above the joint D E; then the moment of the pressure is

$$\left.\begin{array}{ll} H\,P = H\,p\,S & \text{for a square chimney;} \\ H\,P = \dfrac{H\,p\,S}{2} & \text{for a round chimney;} \end{array}\right\} \ldots\ldots\ldots\ldots (2.)$$

* This Article and that on Reservoir Walls, p. 780, are transferred from *A Manual of Applied Mechanics*.

and to this the *least moment of stability* of the portion of the chimney above the joint D E, as determined by the methods of Article 211, should be equal.*

For a chimney whose axis is vertical, the moment of stability is the same in all directions. But few chimneys have their axes exactly vertical; and the least moment of stability is obviously that which opposes a lateral pressure acting in that direction toward which the chimney leans.

Let G be the *centre of gravity of the part of the chimney* which is above the joint D E, and B a point in the joint D E vertically below it; and let the line $\overline{\mathrm{D\,E}} = t$ represent the diameter of that joint which traverses the point B. Let q', as in former examples, represent the ratio which the deviation of B from the middle of the diameter D E bears to the length t of that diameter.

Let F be the limiting position of the centre of resistance of the joint D E, nearest the edge of that joint towards which the axis of the chimney leans, and let q, as before, denote the ratio which the deviation of that centre from the middle of the diameter D E bears to the length t of that diameter.

Then, as in equation 3 of Article 211, the least moment of stability is denoted by

$$\mathrm{W} \cdot \overline{\mathrm{B\,F}} = (q - q')\,\mathrm{W}\,t \ldots\ldots\ldots\ldots\ldots\ldots (3.)$$

The value of the co-efficient q is determined by considering the manner in which chimneys are observed to give way to the pressure of the wind. This is generally observed to commence by the opening of one of the bed-joints, such as D E, at the windward side of the chimney. A crack thus begins, which extends itself in a zig-zag form diagonally downwards along both sides of the chimney, tending to separate it into two parts, an upper leeward-part, and a lower windward part, divided from each other by a fissure extending obliquely downwards from windward to leeward. The final destruction of the chimney takes place, either by the horizontal shifting of the upper division until it loses its support from below, or by the crushing of a portion of the brickwork at the leeward side, from the too great concentration of pressure on it, or by both those causes combined; and in either case the upper portion of the structure falls in a shower of fragments, partly into the interior of the portion left standing, and partly on the ground beside its base.

It is obvious that in order that the stability of a chimney may be secure, no bed-joint ought to tend to open at its windward edge; that is to say, there ought to be some pressure at every point of each bed-joint, except the extreme windward edge, where the intensity may diminish to nothing; and this condition is fulfilled

* These references are to Articles in *A Manual of Applied Mechanics.*

with sufficient accuracy for practical purposes, by assuming the pressure to be an uniformly varying pressure, and so limiting the position of the centre of pressure F, that the intensity at the leeward edge E shall be double of the mean intensity.

It has already been shown, in Article 205, what values this condition assigns to the co-efficient q for different forms of the bed-joints. Chimneys in general consist of a hollow shell of brickwork, whose thickness is small as compared with its diameter; and in that case it is sufficiently accurate for practical purposes to give to q the following values:—

$$\left.\begin{array}{l}\text{For square chimneys, } q = \frac{1}{3};\\[1em] \text{For round chimneys, } q = 1\end{array}\right\} \quad .(4.)$$

The following general equation, between the moment of stability and the moment of the external pressure, expresses the condition of stability of a chimney:—

$$\mathrm{H\,P} = (q - q')\,\mathrm{W}\,t \ldots\ldots\ldots \quad .(5.)$$

This becomes, when applied to square chimneys,

$$\mathrm{H}\,p\,\mathrm{S} = \left(\frac{1}{3} - q'\right)\mathrm{W}\,t;$$

and when applied to round chimneys, .(6.)

$$\frac{\mathrm{H}\,p\,\mathrm{S}}{2} = \left(\frac{1}{4} - q'\right)\mathrm{W}\,t.$$

The following approximate formulæ, deduced from these equations, are useful in practice:—

Let B be the *mean* thickness of brickwork above the joint D E under consideration, and b the thickness to which that brickwork would be reduced, if it were spread out flat upon an area equal to the external area of the chimney. That reduced thickness is given with sufficient accuracy by the formula

$$= \mathrm{B}\left(1 - \frac{\mathrm{B}}{t}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7.)$$

but in most cases the difference between b and B may be neglected.

Let w be the weight of an unit of volume of brickwork; being, on an average, about 112 lbs. per cubic foot, or, if the bricks are

dense, and laid very closely, with thin layers of mortar in the joints, from 115 to 120 lbs. per cubic foot. Then we have, very nearly,

$$\left.\begin{array}{l}\text{for square chimneys, } W = 4\,w\,b\,S;\\ \text{for round chimneys, } W = 3\cdot 14\,w\,b\,S;\end{array}\right\}\ldots\ldots\ldots\ldots (8.)$$

which values being substituted in the equation 6, give the following formulæ:—

$$\left.\begin{array}{l}\text{For square chimneys, } H\,p = \left(\frac{4}{3} - 4\,q'\right)\cdot w\,b\,t;\\ \\ \text{For round chimneys, } H\,p = \left(1\cdot 57 - 6\cdot 28\,q'\right)w\,b\,t;\end{array}\right\}\ldots\ldots (9.)$$

These formulæ serve two purposes; first, when the greatest intensity of the pressure of the wind, p, and the external form and dimensions of a proposed chimney are given, to find the mean reduced thickness of brickwork, b, required above each bed-joint, in order to insure stability; and secondly, when the dimensions and form and the thickness of the brickwork of a chimney are given, to find the greatest intensity of pressure of wind which it will sustain with safety.

The shell of a chimney consists of a series of divisions, one above another, the thickness being uniform in each division, but diminishing upwards from division to division. The bed-joints between the divisions, where the thickness of brickwork changes (including the bed-joint at the base of the chimney), have obviously less stability than the intermediate bed-joints; hence it is only to the former set of joints that it is necessary to apply the formulæ. To illustrate the application of the formulæ, a table is given on page 788, showing the original dimensions and figure, and the stability against the wind, of the great chimney of the works of Messrs. Tennant & Company, at St. Rollox, Glasgow, which was erected from the designs of Messrs. Gordon and Hill, and is, along with Townsend's higher chimney, afterwards described, an interesting example of brickwork used in tall structures.*

Dams or Reservoir-Walls of masonry are intended to resist the direct pressure of water. A dam, when a current of water falls over its upper edge, becomes a *weir*, and requires protection for its base against the undermining action of the falling stream. Such structures are not considered in the present Article, which is confined to walls for resisting the pressure of water only.

In fig. 99, let E D represent a horizontal bed-joint of a reservoir-wall, which wall has a plane surface C D exposed to the pressure

* See p. 788.

of the contained water, whose upper surface is the horizontal plane O Y. Consider a vertical layer of the wall of the length unity, sustaining the pressure of a vertical layer of water of the length unity also. Then from Articles 89 and 124 it appears, that the total pressure exerted against that layer of the wall is equal to the weight of the triangular prism of water O D K, right angled at D, whose thickness is unity, and whose side $\overline{DK}$ is equal to the depth of the joint D E beneath the surface O Y; and it also appears, that the resultant of that pressure acts in the line H C, being a perpendicular upon O D from the centre of gravity H of the prism of water; so that

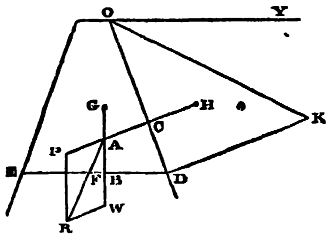


Fig. 99.

$\overline{CD} = \frac{\overline{OD}}{3}$. Let G be the centre of gravity of the vertical layer of masonry above D E, and G B W a vertical line drawn through it; produce H C, cutting that vertical line in A; take $\overline{AW}$ to represent the weight of the layer of masonry, and $\overline{AP}$ to represent the pressure of the layer of water; complete the parallelogram A P R W; $\overline{AR}$ will represent the total pressure on the joint D E for each unit of length of the wall, and F, where that line cuts D E, will be the centre of resistance of that joint, which must fall within the limits consistent with stability of position, while at the same time the angle A F D must not be less than the complement of the angle of repose.

To treat this case algebraically, let x denote the depth of D beneath the surface of the water, w' the weight of an unit of volume of water, and j the inclination of O D to the vertical. Then the pressure of the vertical layer of water is

$$P = \frac{w' x^2}{2} \cdot \sec j, \qquad (1.)$$

its centre C being at the depth $\frac{2}{3} x$.

This force, together with the equal and opposite oblique component of the resistance of the joint D E at F, constitute a couple tending to overturn the wall, whose arm is the perpendicular distance of F from C P; that is to say,

$$\overline{CD} - \overline{FD} \cdot \sin j.$$

Now $\overline{CD} = \frac{x \cdot \sec j}{3}$, and if, as before, we make $\overline{ED} = t$, $\overline{FD} = \left(q + \frac{1}{2}\right) t$; consequently we have for the arm of the couple in question,

$$\frac{x \cdot \sec j}{3} - \left(q + \frac{1}{2}\right) t \cdot \sin j,$$

which being multiplied by the pressure, gives the moment of the overturning couple; and this being made equal to moment of stability of the wall, we obtain the following equation :—

$$\mathrm{W} \cdot \overline{\mathrm{FB}} = \mathrm{W}(q \pm q') \, t = \frac{w' x^3}{6} \cdot \sec^2 j - w' x^2 t \left(\frac{q}{2} + \frac{1}{4}\right) \tan j \ldots (2.)$$

When the inner face of the wall is vertical, $\sec j = 1$, and $\tan j = 0$; and the above equation becomes

$$\mathrm{W}(q \pm q') \, t = \frac{w' x^3}{6} \ldots\ldots\ldots\ldots\ldots\ldots (2 \text{ A.})$$

To obtain a convenient general formula for comparing walls of similar figures but different dimensions, let n, as in Article 211, denote the ratio of the area of the vertical section of the wall to that of the circumscribed rectangle, so that if w be the weight of an unit of volume of masonry, the weight of the vertical layer of masonry under consideration is

$$\mathrm{W} = n \, w \, h \, t,$$

where h is the depth of the joint $\overline{DE}$ below the top of the wall. Then equations 2 and 2 A take the following forms :—

$$n(q \pm q') \, w \, h \, t^2 = \frac{w' x^3}{6} \sec^2 j - w' x^2 t \left(\frac{q}{2} + \frac{1}{4}\right) \tan j ; \ldots\ldots (3.)$$

$$n(q \pm q') \, w \, h \, t^2 = \frac{w' x^3}{6} ; \ldots\ldots\ldots\ldots\ldots\ldots (3 \text{ A.})$$

—equations analogous to equation 4 of Article 213. To obtain a formula suitable for computing the requisite thickness of wall t, let

$$\frac{w' x^3 \cdot \sec^2 j}{6 \, n \, (q \pm q') \, w \, h} = \mathrm{A};$$

$$\frac{w' x^2 \left(\frac{q}{2} + \frac{1}{4}\right) \tan j}{2 \, n \, (q \pm q') \, w \, h} = \mathrm{B};$$

then

$$t^2 = A - 2 B t;$$

which quadratic equation being solved, gives

$$t = \sqrt{A + B^2} - B; \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.)$$

or for a wall with a vertical inner face, for which B = 0,

$$t = \sqrt{A}. \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4 \text{ A.})$$

In most cases which occur in practice, the surface of the water O Y either is, or may occasionally be, at or near the level of the top of the wall, so that h may be made $= x$. In such cases, let

$$\frac{A}{x^2} = \frac{w' \sec^2 j}{6 n (q \pm q') w} = a,$$

$$\frac{B}{x} = \frac{w' \left(\frac{q}{2} + \frac{1}{4}\right) \tan j}{2 n (q \pm q') w} = b,$$

and we have

$$\frac{t^2}{x^2} = a - 2 b \frac{t}{x},$$

which being solved, gives

$$\frac{t}{x} = \sqrt{a + b^2} - b; \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.)$$

and for a wall with a vertical inner face,

$$\frac{t}{x} = \sqrt{a} = \sqrt{\left(\frac{w'}{6 n (q \pm q') w}\right)} \ldots\ldots\ldots\ldots (5 \text{ A.})$$

The vertical and horizontal components of the pressure of the water are respectively

$$\text{Vertical, } P \sin j = \frac{w' x^2}{2} \tan j,$$

$$\text{Horizontal, } P \cos j = \frac{w' x^2}{2};$$

Consequently the condition of *stability of friction* at the joint D E is given by the equation

$$\frac{P \cos j}{W + P \sin j} = \frac{w' x^2}{2 W + w' x^2 \tan j} \leqq \tan \phi \ldots\ldots\ldots (6.)$$

If the ratio $\frac{t}{x}$ has been determined by means of equation 5, then we have

$$\mathrm{W} = n\,w\,x\,t = n\,w\,x^2 \cdot \frac{t}{x}; \quad \ldots\ldots\ldots\ldots\ldots\ldots (7.)$$

so that by cancelling the common factor x^2, equation 6 is brought to the following form:—

$$\frac{w'}{2\,n\,w\,\frac{t}{x} + w'\tan j} \quad \tan\varphi \ldots\ldots\ldots\ldots\ldots\ldots (8.)$$

Example I. *Rectangular Wall.*—In this case $n = 1$; $q' = 0$; $j = 0$; consequently,

$$a = \frac{w'}{6\,q\,w}; \; b = 0;$$

equation 5 A becomes

$$\frac{t}{x} = \sqrt{a} = \sqrt{\frac{w'}{6\,q\,w}}; \quad \ldots\ldots\ldots\ldots\ldots\ldots (9.)$$

and equation 8,

$$\frac{w'}{2\,w\sqrt{\frac{w'}{6\,q\,w}}} = \sqrt{\frac{3\,q\,w'}{2\,w}} \leqq \tan\varphi; \quad \ldots\ldots\ldots (10.)$$

but it is unnecessary to attend in practice to this last equation, which is fulfilled for the greatest values of q that ever occur.

Example II. *Triangular Wall,* with the apex at O.

In this case $\frac{t}{x}$ is the same for every horizontal joint; so that if the thickness be just sufficient for stability at any joint, it will be just sufficient for stability at every other joint. A reservoir-wall whose vertical section is triangular, may therefore be said to be *of uniform stability.*

The value of n for a triangle is $\frac{1}{2}$. With respect to the value of q', that case will be considered in which the inner face of the wall is vertical, so that $q' = \frac{1}{6}$, $j = 0$.

Then by equation 5 A,

$$\frac{t}{x} = \sqrt{a} = \sqrt{\left\{\frac{w'}{3\left(q+\frac{1}{6}\right)w}\right\}}; \ldots\ldots\ldots\ldots (11.)$$

and by equation 8

$$\frac{w'}{w\frac{t}{x}} = \sqrt{\left(3\left(q+\frac{1}{6}\right)\frac{w'}{w}\right)} \leq \tan\varphi \ldots\ldots\ldots (12.)$$

This last equation fixes a limit to the value of q, independently of the distribution of the pressure on each bed-joint, viz.—

$$q \leq \frac{w}{3\,w'} \cdot \tan^2\varphi - \frac{1}{6} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (13.)$$

The insertion of this value of q in equation 11 gives

$$\frac{t}{x} = \frac{w'}{w \cdot \tan\varphi} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14.)$$

The value of $\tan\varphi$ for *masonry* being about 0·74, w being on an average 125 lbs. and w' 62·4 lbs. per cubic foot, the limit of q is found to be

$$0{\cdot}365 - 0{\cdot}167 = 0{\cdot}198, \text{ or } \frac{1}{5} \text{ nearly,}$$

and that of $\frac{t}{x}$, by equation 14, is

$$0{\cdot}674$$

For *brickwork*, $\tan\varphi$ is about the same as for masonry, and w is 112 lbs. per foot, nearly; hence the limit of q is

$$0{\cdot}327 - 0{\cdot}167 = 0{\cdot}16, \text{ or } \frac{1}{6}, \text{ nearly,}$$

while that of $\frac{t}{x}$ is 0·75.

Example III. *Triangular Wall with Vertical Axis.*—When the wall stands on a soft foundation, it may be desirable in some cases so to form it, that the centre of resistance F shall be at the middle of each joint, and shall also be vertically beneath the centre of gravity of the part of the wall above the joint. In this case, the point of intersection A of the lines of action of the pressure and weight must also fall in the middle of each joint. To fulfil these conditions, the vertical section of the wall should be an isosceles triangle, the outer and inner faces forming equal angles j on

opposite sides of the vertical axis of the wall, and the angle j should be such that a straight line perpendicular to O D at O shall bisect the base; that is to say,

$$\frac{t \sin j}{2} = \frac{x \sec j}{3};$$

but

$$\frac{t}{2x} = \tan j,$$

whence we have

$$\sin^2 j = \frac{1}{3}; \; \cos^2 j = \frac{2}{3},$$

$$\tan j = \frac{t}{2x} = \sqrt{\frac{1}{2}} = 0\ 707; \qquad .(15.)$$

and

$$j = 35° \frac{1}{4};$$

so that the base of the wall is to its height as the diagonal to the side of a square.

Equation 8 in this case becomes

$$\frac{w'}{w\sqrt{2} + w'\sqrt{\frac{1}{2}}} = \frac{w'\sqrt{2}}{2w + w'} \leqq \tan \varphi \ldots \qquad .(16.)$$

This condition is always fulfilled so far as the frictional stability of one course of masonry on another is concerned. As the object, however, of giving the wall the figure now in question, is to distribute the pressure uniformly over a soft foundation, let it be supposed that its base rests on a material for which $\tan \varphi = \frac{1}{4}$. Then we must have

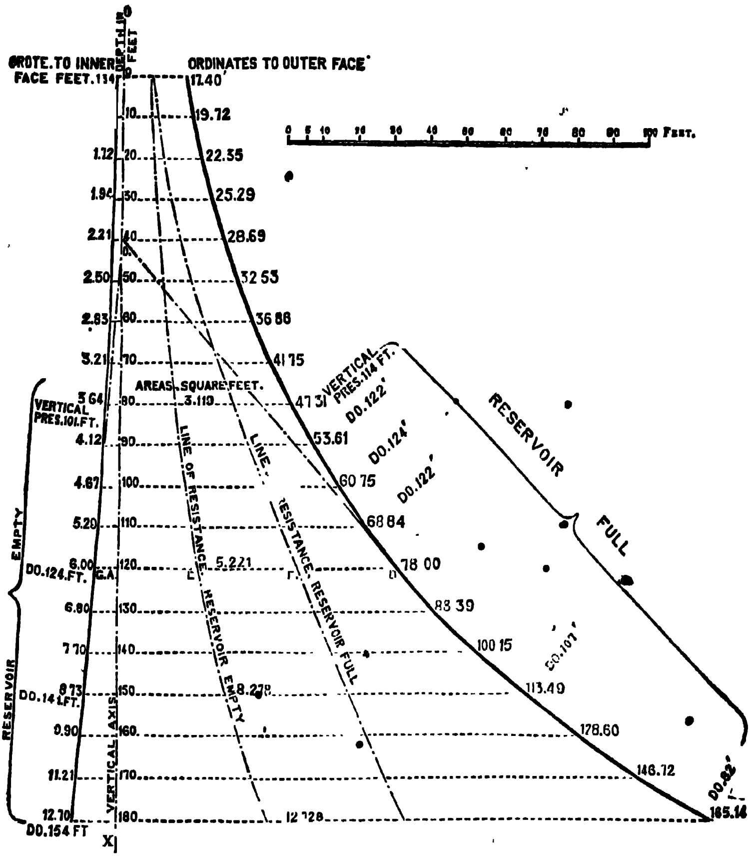
$$\frac{w'\sqrt{2}}{2w + w'} \leqq \frac{1}{4};$$

and consequently

$$w \geqq 2\left(\sqrt{2} - \frac{1}{4}\right) w' = 2{\cdot}33\, w' = 145 \text{ lbs. per cubic foot;}$$

and unless the masonry be of this weight per cubic foot, its friction on a horizontal base, of a material for which $\tan \varphi = \frac{1}{4}$, will not be of itself sufficient to resist the thrust of the water.

The diagram on p. 787 shows the form and proportions adopted by Professor Rankine for reservoir-walls of great height.



For detailed description see *The Engineer* for January 5, 1872 (a reprint of this paper is embodied in the Rankine Memorial Volume of Selected Papers).

The lines of resistance lie within, or near to, the middle third of the thickness of the wall. The outer and inner faces are logarithmic curves. It is desirable to give such walls a curvature in plan convex towards the reservoir, to counteract the tendency of the wall to being bent by the pressure of the water into a curved shape, concave towards the water.

*Dimensions and Stability of the Outer Shell of the Great Chimney of St. Rollox.

Divisions of Chimney.	Heights above Ground.	External Diameters.		Thicknesses.		Greatest pressure of Wind consistent with Security.
	Feet.	Feet.	Inches.	Feet.	Inches.	lb. per square foot.
	435½	13	6			
V.				1	2	
	350½	16	9			77
IV.				1	6	
	210½	24	*0			55*
III.				1	10½	
	114½	30	6			57
II.				2	3	
	54½	35	0			63
I.				2	7½	
	0	40	0			71

Foundation.	Depth below Ground.	External Diameter.	Thicknesses. Concrete.		Brick.
	Feet.	Feet.	Feet.	Inches.	Feet.
I.	0	50	5	0	3
II.	8	50	4	8	3
	14	50			12
III.	20	50	25	0	0

Total height from base of foundation to top of chimney, 455½ feet.

Townsend's Chimney, Glasgow. Built, 1857–59.

Total height 468 feet. Height from surface of ground to cope, 454 feet. Extra height of 20 feet of ornamental iron work since added, and connected with the lightning conductor.

Outside diameter at foundations, 50 feet; outside diameter at surface of ground, 32 feet; outside diameter at top of cope, 12 feet 8 inches. The sides have a straight batter. The thickness varies from 7 bricks at base to 1½ brick at top.

* Joint of least stability.

ADDENDA.

ARTICLE 33, NOTE, p. 47, insert at end of note.

"In surveys of newly settled districts, where it is impracticable to obtain a base by direct measurement with sufficient precision, a base may be measured with accuracy sufficient for ordinary purposes in the following manner:—Choose, as ends of the base, two elevated stations which can be seen from each other, as nearly as possible in the same meridian, and, if possible, 50 or 60 miles asunder; find their latitudes (as explained in Article 86 c, p. 129); also find the true meridian, and the azimuth of the base, from the mean of observations made at each of the stations (as explained in Article 42, p. 71); compute, by equation 1 of this note, the length m of a minute of the meridian corresponding to the *mean* latitude; then *length of base* nearly $= m \times$ difference of latitude in minutes $\times$ secant of azimuth."

ARTICLE 57, p. 91.

LEVELLING BY THE BAROMETER.—To correct the difference of level given by the formula in the text for variations in the force of gravity, multiply by

$$1 + 0{\cdot}00284 \cos. 2\,\lambda + \frac{h}{10{,}450{,}000};$$

in which λ is the mean latitude of the two stations, and h the mean of their heights in feet above the level of the sea.

ARTICLE 110, p. 171.

From some experiments made by Mr. R. D. Napier, forming the subject of a paper on friction and unguents, read before the Philosophical Society of Glasgow, on 16th December, 1874, by that gentleman, and experiments made by an eminent foreigner, but believed to be not yet published, it may be safely deduced that the friction between two bodies is a function of the force with which they are pressed together and of their relative velocity of motion. It is further probable that for substances without unguents, the friction increases with the velocity to a certain maximum, and then diminishes. Mr. Napier believes his experiments show "that with mineral oils the co-efficient of friction is less at higher than at lower velocities, and that with animal and vegetable oils the reverse is the case;" and further, with the employment of unguents the friction has been found to increase with the velocity and *vice versâ*, and also to diminish with the velocity and *vice versâ*. A very small co-efficient of friction was found to obtain when a small "quantity of water was allowed to run on the top of oil." As Mr. Napier points out, there is necessity for further experiment.

In a paper, of which an abstract has appeared in the *Comptes Rendus* of the French Academy of Sciences for the 26th of April, 1858, M. H. Bochet describes a series of experiments which have led him to the conclusion, that the friction between a pair of surfaces of iron is not, as it has hitherto been believed, absolutely independent of the velocity of sliding, but that it diminishes slowly as that velocity increases, according to a law expressed by the following formula:—Let

R denote the friction;

Q, the pressure;

v, the velocity of sliding, in mètres per second = velocity in feet per second $\times$ 0·3048;

f, a, γ, constant co-efficients; then

$$\frac{R}{Q} = \frac{f + \gamma a v}{1 + a v}$$

The following are the values of the co-efficients deduced by M. Bochet from his experiments, for iron surfaces of wheels and skids rubbing longitudinally on iron rails:—

f, for dry surfaces, 0·3, 0·25, 0·2; for damp surfaces, 0·14.

a, for wheels sliding on rails, 0·03; for skids sliding on rails, 0·07.

γ, not yet determined, but treated meanwhile as inappreciably small.

ARTICLE 280, p. 416.

LINE OF PRESSURES IN AN ARCH.—As to the stability of a vertically-loaded arch, reference may be made to a series of papers which have appeared in the *Civil Engineer and Architect's Journal* for 1861, and in which the figure of the line of pressures is shown to be identical with that of a curve whose ordinates represent bending moments.

ARTICLE 357, p. 509.

Mean results of experiments by W. H. Barlow, Esq., F.R.S.:—

	Tenacity. Lbs on the Square Inch.	Proof Strength Transversely Loaded. Lbs. on the Square Inch	Modulus of Elasticity under Transverse Load. Lbs. on the Square Inch.
Puddled steel, specimen I., .	95,233		
,, specimen II.,	116,336	62,500	22,964,000
,, ,, Cast in ingots, . .	101,753		
Puddled steel, specimen III.,		60,000	20,544,000
,, specimen IV.,		63,750	24,802,000
,, specimen V., .		52,500	22,846,400
Homogeneous metal,	100,994	57,500	23,833,600
Steely iron, ...	69,456	52,500	22,846,400

Weight of a cubic foot of puddled steel, 485·5 lbs.; of steely iron, 483·6 lbs. (See the *Engineer* of 3rd January, 1862.)

ARTICLE 357, p. 509.

STRENGTH OF COLD-ROLLED IRON.—The following results were obtained in 1861, through some experiments by Fairbairn on the tenacity of iron. (See *Manchester Transactions*, 10th December, 1861.)

	Tenacity. Lbs per Square Inch.	Ultimate Extension.
Black bar, ...	58,627	·200
Same bar iron, turned, ...	60,747	·220
Same bar iron, cold-rolled, . . .	88,229	·079
Cold-rolled plate, ...	114,912	

Mean results of experiments by M. Tresca, on bars cut out of cast-steel boiler plates:—

	Tenacity. Lbs. on the Square Inch.	Limit of Elasticity. Lbs on the Square Inch.	Modulus of Elasticity. Lbs. on the Square Inch.
Hard steel, untempered, .	74,000	36,000	29,500,000
,, tempered, . .	103,000?	71,900?	27,800,000
Soft steel, untempered, .	81,700	34,100	24,500,000
,, tempered, . .	121,700	105,800	28,800,000

The column headed "limit of elasticity" gives the tension up to which the elongation was sensibly proportional to the load. The results marked (?) are doubtful, because of discrepancies amongst the experiments of which they are the means.

As to the tenacity of wrought iron and steel generally, see Mr. David Kirkaldy's work on that subject, referred to at p. 513.

Article 406, p. 607.

Tubular Foundations.—For excavating the earth inside iron cylinders that are being sunk for foundations, Mr. Milroy introduced the following digging apparatus:—A polygonal iron frame is suspended in a horizontal position by means of chains. It has hinged to it, by their broad ends, a set of triangular, or nearly triangular shovels, which, when they are supported by catches in a horizontal position, with their small ends meeting at the centre of the frame, form a sort of flat tray. When the catches are let go, the shovels hang with their points downwards. In this position they are lowered, and forced into the earth at the bottom of the cylinder. The points of the shovels are then hauled together by means of chains, so as to form the tray, which is wound up with its load of earth by means of a steam windlass; a truck is wheeled upon rails over the mouth of the cylinder, so as to be under the tray; the catches are let go, so as to drop the shovels, and let the earth fall into the truck, which is wheeled away; and the apparatus is ready to be lowered again. By means of this apparatus, cylinders 8 feet 4 inches in diameter, together with the excavation inside, have been sunk at the average rate of about a foot an hour, including stoppages to put on new lengths of cylinder. The numbers of men employed were, one at the winding steam engine, six at rollers for hauling chains to force the shovels into the ground, and afterwards to pull their points together; three at the truck, and one with a shovel and barrow; in all, eleven men; but several of those men might be saved by working the chains from the steam engine.

Article 420, p 628.

Steam Rollers.—Great advantages are derived from steam rolling, and it has come into very general use latterly on economical grounds. It has been found from experience here, but specially on the Continent, that the road wears better, and that there is naturally less wear and tear of rolling stock. On unrolled roads a great deal of the metal is crushed, which is simply worn down by friction upon rolled roads. The objection that was first raised to the introduction of rollers was the injury to subways from their weight. The weight has now been reduced from 36 to 15 tons, which is found to be quite sufficient for all purposes. The economy obtained by their employment is stated to be about 30 per cent., which is the amount that is reduced to dust by traffic before the road is consolidated, on unrolled roads.

Article 434, p. 649.

Engines for drawing heavy loads up steep inclinations are sometimes made with ten or even twelve small wheels, 3 or $3\frac{1}{2}$ feet in diameter, and all coupled so as to act as driving wheels. The lower carriage is jointed, so as to enable it to pass easily round curves: the two sets of axles are sometimes coupled by an ingenious system of link-

driving wheels (according to Mr. Fell's invention) are assisted by a set of horizontal wheels of the same diameter, driven at the same speed, and made, by means of springs, to grasp a high central rail with the tightness required to produce the necessary adhesion.

Article 430, p. 639.

Wire Tramways.—The following description of Hodgson's Wire-rope Transport System is abridged from a published pamphlet on that subject:—

"Lines of this system . . . may be described as consisting of an endless wire-rope, supported on a series of pulleys carried by substantial posts, which are ordinarily about 300 feet apart; but, where necessary, much longer spans are taken, in many cases amounting to 1,000 feet. This rope passes at one end of the line round a drum, driven by a steam engine, or other available power, at a speed of from four to eight miles an hour. The boxes in which the load is carried are hung on the rope at the loading end, the attachment consisting of a pendant of peculiar shape, which maintains the load in perfect equilibrium, and at the same time enables it to pass the supporting pulleys with ease. Each of these boxes carries from 1 cwt. to 10 cwt., and the delivery is at the rate of about 200 boxes per hour for the entire distance. . . . A special

arrangement is made at each end of the line, consisting of rails so placed as to receive the small wheels with which the boxes are provided, and deliver them from the rope. The boxes thus become suspended from a fixed rail instead of the moving rope, and can be run to any point to which the rail is carried, for loading or delivering, and again run on to the rope, for returning. The succession is continuous, and the rope is never required to stop. . . . Curves are easily passed, and inclines of 1 in 6 or 7 are admissible. . . . The rope being continuous, no power is lost on undulating ground."

Article 435, p. 656.

Narrow Gauge Railways.—Railways of gauges smaller than that commonly called the "narrow gauge" are used where the traffic is light, and cheapness of first cost is important. Some Norwegian railways have a gauge of $3\frac{1}{2}$ feet.

The Festiniog Railway in North Wales has a gauge of only 2 feet. The rails weigh 30 lbs. to the yard, and are in lengths of 18 and 21 feet. The intermediate chairs weigh 10 lbs.; the joint chairs, 13 lbs. The sleepers are of larch, 4 feet 6 inches long, from 9 to 10 inches broad, and from $4\frac{1}{2}$ to 5 inches deep. At each side of a joint they are 1 foot 6 inches apart from centre to centre; elsewhere, 2 feet 8 inches. Clear width of roadway for a single line, 12 feet; central space of a double line, 7 feet; clear width, 21 feet. Sharp curves, from 2 to 4 chains radius. Steepest gradient on passenger line, 1 in 80 nearly; elsewhere, 1 in 60. Passenger carriages 10 feet long, 6 feet 3 inches wide, 6 feet 6 inches high; four wheels, 1 foot 6 inches diameter; wheel base, 4 feet; carry ten passengers, in two rows, back to back. Engine weighs when full, $7\frac{1}{2}$ tons; four wheels, coupled, 2 feet diameter; wheel base, 5 feet. Two outside cylinders, 8 inches diameter, 12 inch stroke; greatest steam-pressure, 200 lbs. on the square inch above atmosphere. Water carried in a tank over the boiler; fuel in a 4-wheeled tender. The ordinary speed ascending 1 in 80, with a gross load of 50 tons, exclusive of engine and tender, is 10 miles an hour. As to "Fairlie" engines, which are well suited for narrow gauge railways, see p. 649.

Article 445, p. 673.

Extensive and numerous experiments have been recently made by Mr. Robt. Gordon of the P. W. D., India, on the Irrawaddi, and by Messrs. Humphreys & Abbot on the Mississippi, on the vertical distribution of velocities in a current. These, and some other experiments of less magnitude, are discussed by M. Bazin in No. 36 of the *Annales des Ponts et Chaussées* for 1875, p. 309. The velocities taken on the same vertical vary as the ordinates of a parabola; the greatest velocity is generally at the surface, but sometimes below it. The following formula is proposed:—

$$v = V - M\,(x - a)^2; \quad \text{(1.)}$$

in which v is the velocity at the depth h, which bears to the total depth H the ratio x, V is the maximum velocity, M is the parameter, a is the ratio of the distance h' of the summit of the parabola from the surface to the total depth H. The mean velocity is—

$$u = V - M\,(\tfrac{1}{3} - z + z^2); \quad \text{(2.)}$$

and the velocity at half depth $v_{\frac{1}{2}} = u + \frac{M}{12}$ the value of M is $20\sqrt{H\,I}$, where I is the inclination of the bed.

Both M. Bazin and Mr. Gordon consider that the relative velocities at different portions of a given section are influenced by the relative roughness of the bed, but the proportions have not yet been definitely fixed by formulæ. Mr. Gordon considers further that these relations are affected by the depth of water in the same stream, and by the distance or position of the float from the sides.

Article 498, p. 738.

Filtration.—From experiments by M. Havrez (*Revue Universelle des Mines*, May, June, 1874, and Vol. 39, *Proc. Inst. C.E.*), it appears that filtration is influenced by the pressure and temperature of the water, the thickness of the filtering medium, the size of the grains forming the filter, and their mixture. The delivery of

a filter per square foot per twenty-four hours, is equal to 2·4 cubic yards, multiplied by the pressure of the water in yards, and divided by the thickness of the filtering medium in yards nearly. When large and small grains of sand are mixed the delivery is found to diminish, as also by silting and fouling. Formulæ for the velocity influenced as stated above, are given in the original paper, to which reference may be made.

For filtration the upper part of the sand should be pared off before it gets clogged, and placed inside a box to be washed. The box is made with a false bottom with small holes, like the tiles used below the sand in the filter beds. Water under pressure is allowed to flow into the space between the bottom of the box and the false bottom, this water, in rising up through the holes, causes the sand to be stirred, and the water carries off the dirty particles at the top and runs away by an overflow. A suitable arrangement of materials for a filter is as follows, and arranged in the order given:—Sand, 2 to 3 ft.; fireclay tiles 10″ × 10″ with 50 holes each, 2 ins; gravel, 6 ins. to 1 ft.; broken stone, 1 ft. 6 ins. to 2 ft. The area of filter bed may be about 1 sq. yard to every 700 galls. required per day.

From Dr. Frankland's experimental investigations it appears that the bacteria which may be present in water can be removed to a large extent by sand filtration, and for this reason slower rates of filtration are recommended, varying with the nature of the water. The depth and fineness of the sand is of importance.

Article 230, p. 373.

Concrete is now largely used in engineering works: such as in harbour and dock walls, reservoir walls, foundations, &c. It is used either in blocks, made by placing the newly mixed concrete in wooden frames, or in monoliths by erecting a casing composed of planks arranged to the form and height of wall required, the concrete is then placed and spread layer by layer within the casing. On the removal of the boarding, the exposed faces receive a coat of cement mortar. In some works recently executed, blocks of from 70 to 350 tons have been used. Concrete in bags has also been used successfully for foundation work in sea walls

The essential requisites in obtaining good concrete are strong hydraulic lime or cement, and clean sharp sand; the stones should also be clean and free from earthy matter. Various proportions are specified by engineers according to the nature of the work. If Portland cement is used it should be fresh, and should stand a tensile stress of from 300 to 350 lbs. per square inch, after having been made into test bricks, and immersed in water seven days. The weight of the cement should be about 112 lbs. per bushel. Where strong compact concrete is required the proportions are usually about 1 of Portland cement and 1 of sand, with 4 or 5 parts of broken stone from 2 to 2½ inches in size.

Articles 356 and 439, pp. 506 and 665.

Steel is now largely used for rails, tires, ship and boiler plates, as also for propeller blades, and for hollow propeller shafts. In the Bessemer process a large pear-shaped vessel, called the converter, capable of holding three or four tons, composed of wrought-iron plates rivetted together and lined with gannister, is filled with molten pig-iron from the cupola, or, in some cases, is filled direct from the blast furnace; a blast of air is then driven through the liquid mass, by which the silicon and carbon are oxidised. This process, which lasts for about twenty minutes, is accompanied by intense heat, with a brilliant display of flame and sparks from the mouth of the converter; and the elimination of the carbon, &c., is usually determined by the eye as taking place when the flame drops and becomes of a brownish tinge, it may also be determined by the spectroscope through the disappearance of the carbon lines. The requisite addition of carbon and manganese is then added by running into the converter a quantity of melted spiegeleisen or ferro-manganese, and the whole is then poured into cast-iron moulds

In the Siemens or Siemens-Martin process a mixture of iron, scrap, and ore is melted in an open hearth furnace by means of gas supplied by gas producers, and worked upon the heat regenerative system. After the silicon, carbon, &c., are oxidised and addition of the spiegeleisen or ferro-manganese, it is poured into cast-iron moulds. The charge is about ten tons, and the whole process takes several hours.

The ingots of cast steel are thereafter re-heated, hammered, and taken to the rolling mill to be turned out as plates. Rails are rolled direct from the reheated ingot. The steel produced in this manner is known as "mild steel," and possesses wonderful strength and ductility. From a careful investigation of its properties the Committee of Lloyds have adopted the following tests for ship's plates.

Ultimate tensile strength 27 to 31 tons per square inch.

Elongation 20 per cent. on a length of 8 inches before fracture.

Strips heated to low cherry-red, and cooled in water at 82° Fahr., must stand bending double round a curve of diameter not more than three times thickness of plate.

About 28 tons per square inch appears to be about the average strength, with the limit of elasticity about one-half of this, and the elongation about 25 per cent.

The Board of Trade have now allowed the use of steel for bridge structures, with a working strength of $6\frac{1}{2}$ tons per square inch.

By the use of steel in ships and bridges a saving in weight is effected of from 16 to 20 per cent. The usual test for steel rails is that of a 1 ton weight falling from a height of 5 feet, or a load of 20 tons resting mid-way on bearings 3 feet apart.

It is advisable to drill the rivet holes in steel plates, as punching reduces the strength by about 30 per cent.; the strength, however, appears to be restored by annealing.

The shrinkage of cast steel is about double that of cast iron.

Various articles, such as wheels, axles, propellers, &c., are made of cast steel; these have afterwards to be annealed to insure uniformity of strength.

Heavy steel plates have been used for armour plating for war ships, and recent experiments seem to show an advantage in using a combination of steel and iron, the steel plate being in front. As the presence of phosphorus is detrimental to the production of steel, non-phosphoretic ores have been almost exclusively used. Various attempts have, however, been made from time to time to make use of the more phosphoretic ores, and recently considerable success appears to have accompanied experiments in this direction, and known as the "Thomas-Gilchrist" process. In this method the converter is lined with a basic material instead of the ordinary siliceous one, and at certain stages of the process lime is added; the result being that the phosphorus unites with the basic material and leaves the iron comparatively free.

Articles 429, 430, and 433, pp. 635, 640, and 647.

Weights of Locomotives and Carriages.—The weights of passenger locomotives are now 40 tons and upwards, with a load on driving wheels of $15\frac{1}{2}$ tons, the steam pressure being about 175 lbs to the square inch. Diameter of cylinders 17 inches. Stroke of piston 24 inches. Heating surface about 1,140 square feet. The weight of tender is about 32 tons. Size of driving wheels from 7 to 8 feet.

In goods locomotive engines weighing about $37\frac{1}{2}$ tons and having six wheels coupled, the load on driving wheels is about $13\frac{1}{2}$ tons, with stroke of piston of 26 inches, the pressure of steam and weight of tender being same as for passenger engines.

For very heavy traffic engines are used having cylinders of 19 inches diameter, stroke 26 inches, heating surface 2,000 square feet, and steam pressure 200 lbs.: the weight being 70 tons. Carriages weigh from 10 to 20 tons.

Article 431, p. 641.

A mountain railway may be defined as one in which the gradient is steeper than 1 in 50. It may be worked either by a fixed engine or locomotives. The former plan is adopted for short lengths on lines with easier gradients, the latter for longer lines. Numerous methods have been employed on different lines for working steep gradients; the most usual being a rack rail laid down between the ordinary rails, in which gears a tooth wheel driven by a locomotive, and V form guide rails placed at short distances along the centre of the way, in which works a horizontal drum, fixed to the locomotive, with threads running in opposite directions from the centre. On the other hand, gradients of 1 in 14 have been worked, both in ascent and descent, with ordinary locomotives provided with powerful brakes, the wheel base and diameter of the wheels being diminished. Engines suitable for steep gradients are not suitable for sharp curves, and the powerful engines required, on the necessarily short wheel base, cause unsteady action, whilst the great weight has an injurious lateral effect and grinding

action on the wheel, and there is great difficulty in keeping the tubes and fire-box of the boiler covered with water. So far, experience has seemed to be in favour of engines on bogies of the Fairlie type.

In the Righi Railway, Switzerland, the gradient is about 1 in 4. There is a rack rail in centre. The engine has a vertical boiler. Gauge 1 mètre.

The Vesuvius Railway is worked by a fixed engine and wire ropes. The inclination of the line is about 50°.

ARTICLE 477, p. 721.

CAST-IRON WATER PIPES.—It is usual to cast the large sizes, and most of the smaller sizes, of such pipes vertically, the faucet end being downwards; in some cases the smaller sizes are cast on a slope, slightly inclined to the horizontal.

The lengths of pipes vary from 12 to 6 feet exclusive of the faucet.

The pipes are coated by dipping them into pans containing a boiling composition of pitch and oil.

The "turned and bored" joint is largely used; the parts to be fitted being turned to gauges of a suitable taper (about 1 in 32). Anticorrosive paint or Portland cement is used for coating the turned parts before driving home.

ARTICLES 366 and 377, pp. 520 and 548. See also ARTICLE 158, p. 237.

The following notes on American Bridge Practice are taken from *The Transactions of the American Society of Civil Engineers*, Vol. VIII.:—

"American bridges are generally built up from the following individual members, most, if not all the mechanical work upon them being done in the shop. 1st. Chord and web eye-bars; round, square, or flat bars, with a head at each end, formed by some process of forging. These are tension members. 2nd. Lateral, diagonal, and counter rods. 3rd. Floor-beam hangers. 4th. Pins. 5th. Lateral struts. 6th. Posts. 7th. Top chord sections. The last three being columns formed by rivetting together various rolled forms; plates, angles, channels, I beams, &c. Some are square-ended, others pin-connected. These are compression members. 8th. Floor-beams and stringers. These consist either of rolled beams, rivetted plate girders, or occasionally of latticed or trussed girders. The proportion of depth to span in American bridges is from one-fifth to one-seventh.

"In top chords, posts, and struts the strains are calculated by a modification of Rankine's formula, as follows:—

$$p = \frac{8000}{1 + \frac{l^2}{40000\, r^2}} \text{ for square-end compression members.}$$

$$p = \frac{8000}{1 + \frac{l^2}{30000\, r^2}} \text{ for compression members with one pin and one square end.}$$

$$p = \frac{8000}{1 + \frac{l^2}{20000\, r^2}} \text{ for compression members with pin bearings.}$$

where p = the allowed compression per square inch of cross section.
l = the length of compression member, in inches.
r = the least radius of gyration of the section, in inches."

ARTICLE 430, p. 635.

TRACTION ENGINES are now much used for the haulage of heavy articles, such as boilers and machinery. In India they have been successfully used for hauling goods

and passenger trains along the roads, at speeds varying from four to eight miles per hour. The general design adopted is a vertical boiler and geared engines, the whole resting on a car carried by three wheels, the front wheel being used for guiding the engine. The wheels are fitted with India-rubber tires, protected from wear by iron shoes.

Article 410, p. 614.

Dredging.—The large dredgers, as now used on our rivers, will lift about 500 tons of material per hour.

Barge loading dredgers of 60 to 70 nominal H.P., dredging from 6 to 30 feet of depth, will lift about 3000 tons of material per day.

Hopper dredgers of 100 nominal H.P., dredging at same depth, will lift about 2000 tons of material and *deposit* same 6 to 7 miles off per day.

Article 431 a, p. 644.

Continuous Brakes for Railway Trains.—The use of brake power is now considerably extended in railway traffic, and instead of the brakes being only applied on tender and guard's van, the application has been extended to the carriages composing the train. Very considerable resistance is thus obtained, and consequent cessation of motion at a much earlier period. Various forms of continuous brake have been tried recently, and the results of the experiments are familiar to engineers. Some of the various forms are the screw-brake, chain-brake, vacuum-brake, hydraulic-brake, and compressed-air-brake, in all of which, by means of mechanism extending below the carriages and actuated by the engine-driver or guard, the whole or part of the wheels of the train can be braked. In the first two methods, rigid or flexible bodies are employed to transmit the power required, whilst, in the others, the same object is accomplished through the medium of fluids. In the hydraulic-brake, water at a high pressure from a pump on the engine is conveyed by a pipe; in the vacuum-brake the air is removed, and in the air-brake the air is forced under pressure to the points required. In the automatic arrangements, whether of air or vacuum, there are reservoirs. It has been found desirable to adopt reservoirs or vessels having pistons immediately in connection with the brake blocks, the object in the automatic arrangements being to keep up a certain condition in the chambers, whether of pressure or vacuum, by which, if destroyed either intentionally or accidentally (as through the breakage of a pipe), the braking action may at once take place.

In some cases 1½ seconds is sufficient to apply the brakes, and fast trains can be stopped in about 300 yards.

Article 207, p. 844.

Dynamite, a pasty substance composed of nitro-glycerine and an absorbent earth, is now largely used as an explosive. It is especially effective in breaking up large stones, blocks of metal, roots of trees, &c., and is of great service for operations under water. Smaller bore holes are required than for gunpowder, and only loose tamping, such as sand or water, is employed. In many cases, such as in breaking up boulder stones, the cartridge is simply laid on the top of the stone, and covered with clay or sand. It burns when in a loose state on the application of a match, and explodes with great violence when fired by a detonating fuse.

From experiments by Major Morant, R.E., India, it appears that only one-half the quantity of dynamite, and one-third of the number of bore-holes, is required to remove the same quantity of rock as when gunpowder is used. For quarrying purposes gunpowder appears, however, to be preferred to dynamite, as having less shattering effect on the rock.

One great advantage in using dynamite is that in many cases it can be used without bore-holes, and when these are used very shallow ones are found sufficient, and do not require tamping; with deep holes, clay or sand, tamping appears most suitable.

ARTICLE 110, p. 171.

COEFFICIENT OF FRICTION.—From experiments made by Capt. Douglas Galton C.B., F.R.S., on the effect of brakes upon railway trains, it appears that

(1.) The retarding effect of a wheel sliding upon a rail is not much less than when braked with such a force as would just allow it to continue to revolve, the distance due to friction of the wheel on the rail being only about $\frac{1}{3}$ of the friction between the wheel and the brake blocks.

(2.) The coefficient of friction between the brake blocks and the wheels varies inversely according to the speed of the train; thus, with cast-iron brake blocks on steel tires, the coefficient of friction when just moving was ·330,

At 10 miles	per hour	·242		At 40 miles	per hour	·140
,, 20	,, ,,	·192		,, 50	,, ,,	·116
,, 30	,, ,,	·164		,, 60	,, ,,	·074

ARTICLE 521, p. 763.

BREAKWATERS.—Concrete enters largely into the formation of breakwaters and quay walls, and is made up into blocks of various sizes, or in masses contained in bags; it is also formed into specially moulded sections or cylinders for quay walls.

It is important, where breakwaters are concerned, that the blocks should be large and carefully set, so that the action of the waves and the air during the surging action may have no crevices or open joints to work into.

Many devices have been resorted to to give the building a monolithic character. A good foundation is first of all to be obtained and the blocks carefully set thereon. In some instances, the blocks have been set on the clay of the bottom after the sand had been removed, in others concrete in bags was deposited from a hopper barge, and these allowed to take there own position, and again, masses of rubbish have been laid down, upon which the large concrete blocks were set. (See a paper by Mr. M. Scott, *Proceed. C. Eng.* for 1858; also *Trans. Inst. Engineers and Shipbuilders in Scotland*, vol. xxxvi., 1892-93.)

The blocks are in many cases set with a slope, so as to break the force of the waves, and in some cases are moulded sufficiently broad so as to stretch across the whole width of the breakwater; in other cases there are two or more blocks to the width, but the joints in the interior of the work are sometimes a source of weakness.

The blocks may be joggled together, but in this case there must be a good fit to prevent movement; dowels of iron are also resorted to to bind the work together. The weights of blocks used in some recent examples of breakwaters abroad varied from about 20 tons to 60 tons; the slopes at which the blocks were set varied also, 1 in 4 and 1 in 6 being in some cases adopted.

The proportions of the materials used in such concrete has sometimes been as follows:—

Portland cement, ...	1 part.
Sand, ...	$2\frac{1}{2}$,,
Broken stones, $2\frac{1}{2}$ ins. gauge, ...	$4\frac{3}{4}$,,

ARTICLE 410, p. 615.

Some of the dredgers used on the River Clyde measure 161 feet long by 23 feet broad and 10 feet deep, and have engines of 75 nominal horse-power. They can dredge in 28 feet of water and are fitted with a single bucket ladder. With such a machine upwards of 400,000 tons of material have been dredged during a year's work, extending to about 2,700 hours.

Some of the double bucket ladder dredgers measure 156 feet long by 32 feet broad and 10 feet deep, the engines being 50 nominal-horse power, and can dredge in 32 feet of water. A year's work of about 2,500 hours gives about 533,000 tons of material raised.

ARTICLE 485, p. 656.

The gauge or distance between the rails as now used, is somewhat varied; thus, in England and Scotland it is usually 4 feet 8½ inches, although the Festiniog Railway in Wales has a gauge of 1 foot 11½ inches. In Ireland it is 5 feet 3 inches; in Spain 5 feet 6 inches; in India 2 feet 6 inches, 3 feet 3⅜ inches (*i.e.*, metre gauge), and 5 feet 6 inches; and recently in Canada a railway has been constructed of 3 feet 6 inches gauge—the rails weigh 56 lbs. per yard, and are flat-bottomed, being fastened to the sleepers by spikes. The gauge in the United States is principally 4 feet 8½ and 4 feet 9 and 5 feet.

ARTICLE 364, p. 519.

From a comparison of the strength of iron and steel wire ropes, it appears that the steel rope is nearly double the strength of the iron rope. The following formula appears to give the breaking strength of wire ropes sufficiently accurately for practical purposes—

$$L = 14 \times \left(\frac{C}{3}\right)^2 \text{ for iron wire ropes}$$

$$L = 25 \times \left(\frac{C}{3}\right)^2 \quad \text{, st}$$

Where L = breaking load in tons, and C = circumference of rope in inches.

When the speed of working is high, *one-tenth* of the breaking load should be taken as the working load.

The same rule, as applied to hemp ropes, gives

$$L = 2\tfrac{1}{4}\left(\frac{C}{3}\right)^2$$

TRANSPORTER BRIDGES.—These bridges are designed partly on the suspension principle, so far as the carrying of the load is concerned. The travelling load does not, however, as in the suspension bridge, pass along a platform supported from the cables stretched from tower to tower, but consists of a car suspended from a trolley which runs on rails fixed to a girder stretching across between the towers at a sufficiently high level to permit of shipping passing beneath. The cars are sufficiently large to carry several hundreds of passengers at a time, as well as vehicles. Electric motors are used for the propulsion of the trolley.

ARTICLE 433, p. 645.

LOCOMOTIVES.—From experiments on locomotives* Mr. D. Drummond has found that there is a decided gain in the use of increased pressure—thus, in the case of two similar engines, one of which used steam of a pressure of 175 lbs. and the other of 200 lbs., the following results were obtained:—The weight drawn was, in each case, 217 tons. With the lower pressure the speed was 46·207 miles per hour; the coal per mile was 47·120 lbs., and the water evaporated per lb. of coal was 5·835 lbs. With the higher pressure, the speed was 49·601 miles; the coal 41·029 lbs., and the water 6·945 lbs. The engines had cylinders of 18 inches diameter and 26 inches stroke. The total horse-power runs about 1,000, of which about three-fourths is effective.

* See *Minutes Proceed. Inst. C.E.*, vol. cxxvii.

ARTICLE 150, p. 227.

CORRUGATED FLUES.—From recent experiments on steel corrugated flues by Mr Parker, of Lloyds', the following formulæ are proposed for strength :—

$$\text{Ultimate crushing strength in lbs. per square inch} = \frac{60,000 \times t}{d},$$

where t is the thickness of plate, and d the mean diameter of furnace.

$$\text{Working strength in lbs. per square inch} = \frac{1000 \times (T - 2)}{D},$$

where T is the thickness of plate in sixteenths of an inch, and D the greatest diameter of the furnace in inches. With the latter rule the margin of safety appears to be fully 5.

The experiments were carried out with a flue having a corrugation $1\frac{1}{2}$ inch deep and 6 inches apart. The steel plate showed a tenacity of 22·7 tons per square inch, with an elongation of 35 per cent. in a length of 10 inches.

ARTICLE 330, p. 492.

PRESERVATION OF IRON.—Recently some new processes have been developed for the preservation of iron from rust. One, known as Barff's process, consists in placing the articles in an air-tight oven, where they are heated to a cherry red, a current of superheated steam is then allowed to play upon them, and through its decomposition the oxygen uniting with the iron forms a coating of magnetic oxide of iron. Bower's process appears to attain the same result through the medium of carbonic oxide gas and air at a high temperature.

ARTICLE 388, p. 585.

MANGANESE BRONZE.—The strength and freedom from corrosion of manganese bronze constitute it an excellent material for screw propellers, as blades of lighter section can be used than are required for steel.

The transverse strength appears to be about double that of gun-metal and steel, and its tensile strength varies from 24 to 40 tons per square inch.

ARTICLE 439, p. 665.

NICKEL STEEL.—This steel gives a high tensile strength and limit of elasticity. Its ultimate strength being about 50 tons, and elastic limit 28 tons. Carbon mild steel giving about 28 tons ultimate and 14 tons elastic strength.

ARTICLE 459, p. 699.

WATER METERS.—Kennedy's water meter has the pistons made of vulcanite, and the water ways are protected from corrosion by treatment with Barff's oxidising process, for which see p. 793.

ARTICLE 477, p. 720.

WATER PIPES.—When the diameter of the pipe is great, say 3 feet or 4 feet, it is advisable to cast the lengths without sockets and connect them by a collar of steel plate rolled to the required size, a lead space being provided for. Where sockets are used on such large sizes it is of advantage to shrink on the sockets rings or hoops of iron or steel.

ARTICLE 430, pp. 635, 639.

CABLE TRAMWAYS.—Cable tramways have now been used with successful results in some of the large cities in the United States and Europe.

In San Francisco an endless steel wire rope, $3\frac{1}{2}$ inches in circumference and 11,000 feet long, is supported on sheaves 11 inches in diameter, placed 39 feet apart, the

whole lying in an iron tube placed between the car rails and beneath the roadway, which an opening of about three-quarters of an inch wide is left.

Through this opening the connection between the car is made, and consists of an arm, which grips the rope when the car is to be set in motion. The rope is kept constantly going by means of a powerful steam engine. Additional sheaves are arranged at changes of direction of the rope.

The rope passes round grip pulleys at the engine house, 8 feet in diameter.

The cars are said to be more easily started and brought to rest than when drawn by horses. The stoppage is effected by allowing the "grip" to slacken its hold of the rope, and putting on the brakes

In Chicago about 20 miles of these tramways are laid down, some of the ropes being over 27,000 feet in length. These ropes are made of steel $1\frac{1}{4}$-inch in diameter and have a breaking strength of 39 tons.

Strength of Leather Belts.—Ultimate strength about 3200 lbs. per sq. inch; working strength about one-eighth of this.

2-inch cotton rope, working strength = 600 lbs.

Article 364, p. 519.—Strength of Wires.

Pure Copper wire,	about 18 tons per sq. inch.
Phosphor bronze,	,, 45 ,, ,,
Steel wire,	90 to 140 ,, ,,

Article 378, p. 560.

The cantilever form of bridge, which has been tried on several occasions in America, and has now on a great scale been exemplified in the Forth Bridge, is an economical form of bridge for large spans—viz., those exceeding 600 feet—where scaffolding cannot be erected to build the structure.

The Niagara cantilever bridge crosses the Niagara River about two miles below the Falls, and is close to the Railway Suspension Bridge (illustrated at p. 581). The river gorge here is about 850 feet wide and 210 deep to surface of river, which flows at that part at the rate of about 16 miles an hour. The bridge was erected in eight months. Each cantilever measures 395 feet in length, whilst the intermediate span is 120 feet.

The tower posts, lower chords, centre and end posts of cantilevers, pins, and top castings for towers are of steel, the other parts are mainly of wrought iron.

The towers are 132 feet high, and are composed of four main posts suitably braced

The upper chords of the shore arms receive alternate tensile and compressive strains.

The bridge was designed for the following live loads :—

A train on each track, the engine having 72,000 lbs. on three pairs of drivers, spaced 6 feet between centres, the train weighing 1 ton per lineal foot.

A wind pressure of 30 lbs. per square foot, on a train surface of 10 sq feet per lineal foot of bridge, plus the same pressure per sq. foot upon the vertical surface of one side of the structure and tract.

Article 378, p. 560.

The tensile strains allowed for live loads in the iron members are—10,000 lbs. per sq. inch for main ties and chord bars; laterals and sway bracing, 15,000 lbs ; chord bars for wind strains = 12,500 lbs.

In compression the allowed strain shall meet the requirements of Gordon's formula.

In steel members the tensile strain per sq. inch allowed shall be as follows:—

$$11{,}000\left(1 + \frac{3}{4}\,\frac{\text{minimum strain}}{\text{maximum strain}}\right)$$

This result shall be used in Gordon's formula for compression as the numerator of the fraction in place of 8'00.

The following is a comparison of weights for single span girders, the spans being similar, and for the cantilever system as used in Niagara Bridge:—

Two 208 feet spans, 2800 lbs per lineal foot,	582 tons.
One 495 feet span, 6200 ,, ,,	1534 ,,
	2216 tons.

For Cantilever,	1606 tons
„ Intermediate span,	147 „
„ Anchorage,	90 „
	1843 tons.

The dead load in the Niagara Bridge is 45 per cent. of the total load.

The modulus of elasticity for the wrought iron was taken at 26,000,000, and for the steel at 29,000,000 lbs per sq. inch.*

ARTICLE 384, p. 584.

The tensile strength of lead for pipes, as determined by Kirkaldy, is 2159 lbs. per sq. inch. That this is a good average value appears from the results of experiments on the strength of lead pipes of various diameters and thickness. See annexed Table—

Diameter.	Thickness.	Bursting Pressure	Calculated Tenacity.
Inches.	Inches.	Lbs. per sq. in.	Lbs per sq. in.
$\frac{1}{2}$	·20	1820	2275
	·20	911	2277
$1\frac{1}{2}$	·29	812	2100

The tenacity, as given in the fourth column, is calculated by the formula for strength of pipes $f = \frac{pr}{t}$. (See Art. 150, p. 227.)

ARTICLE 408, p. 611

The caissons used for the foundations of the Forth Bridge (see p. 542*a*) are of great size, being 70 feet in diameter at bottom and 60 feet at low water level, they were sunk to depths of 70 feet to 90 feet, through the softer parts of the bed of the river, until stiff clay or rock was met.

The sinking was accomplished by the pneumatic process, an air chamber about 7 feet high at bottom, in which the men worked under varying pressures, with suitable air locks at top being used.

The pressures required to keep out the water did not, however, necessarily vary with the head of water outside. No inconvenience was felt by the workers so long as the pressure did not exceed about 18 lbs. per sq. inch. Above this sickness was experienced, the ill effects being more noticeable on passing out of the air chamber. Authorities on this subject, therefore, suggest that the change from the greater air pressure of the chamber to the open air should be made slowly.

The caissons were formed of a steel shell, having a steel cutting shoe at bottom, the whole being filled up with concrete and capped by granite masonry.

Groups of four of these constitute the main piers on which the superstructure of the bridge rests.

The excavation was mainly done by hydraulic power, the working chamber being lighted by electricity.

ARTICLE 428, p. 633. (See p. 810.)

The Elevated Railway in New York is carried along some of the principle streets, and is arranged for both single and double traffic. At first it was intended to work it as a cable road, but that being found impracticable locomotives were adopted. The road is carried on iron posts, made up either of channel irons braced or of hollow iron posts filled with cement mortar. These posts are fixed on the outside of the pathway, about the line of the kerbstones, and are placed about 40 ft. apart. Girders of the zig-zag or Warren type rest upon them, on which the sleepers are laid. The locomotives and carriages are small, the latter being seated and arranged like a tram-car. The gauge is 4 ft. $8\frac{1}{2}$ in.

* See *Trans. American Society of Civil Engineers*, 1885.

ARTICLE 430, p. 639.

The weight of American locomotives for goods traffic is heavier than for the same class in this country. Goods engines weigh as much as forty-eight tons, having eight coupled driving-wheels of 50 inches diameter. The largest driving-wheels in use for passenger engines are 6 feet 6 inches coupled. The driving-wheels are commonly made of cast-iron, with steel tires pressed or shrunk on.

The following are some of the leading dimensions of typical passenger engines of British and American design:—

	BRITISH.		AMERICAN.	
Diameter of Cylinder,	18 in.		18 in.	
Stroke,	24 in.		24 in.	
Dia. of Driving-wheels coupled,	7 ft. 2 in.		6 ft. 6 in.	
,, of Bogie-wheels,	3 ft $4\frac{1}{2}$ in.		2 ft. 9 in.	
Wheel base,	21 ft. $2\frac{1}{2}$ in.		22 ft. $7\frac{1}{2}$ in.	
Heating Surface in Tubes,	905 sq ft.		1205 sq. ft.	
,, in Fire-box,	82 sq. ft.			
Fire-grate Area,	14 ft. 7 in.		35 sq. ft for Anthracite coal.	
Diameter of Tubes (Copper),	$1\frac{3}{4}$ in.		$1\frac{5}{8}$ in. wrought iron.	
Boiler-shell—Yorkshire Iron,	$\frac{1}{2}$ in.		$\frac{3}{8}$ in. steel.	
	Tons.	Cwts.	Tons	Cwts.
Weight on Bogie,	13	0	12	4
,, on Driving-wheels,	14	15	15	0
,, on Trailing-wheels,	13	12	14	3
	41	7	41	7

The tractive force is from 90 to 100 lbs. per pound of effective pressure on piston. The tractive force in lbs. of a locomotive engine may be found by the rule $T = \frac{d^2 l p}{D}$.

Where d = dia. of cylinders in inches.
l = stroke in inches.
p = effective pressure of steam in lbs. per sq. inch.
D = dia. of driving-wheels in inches.

ARTICLE 512, p. 753.

During storms in the North Atlantic waves sometimes extend to 500 or 600 feet and last from 10 to 11 seconds.

The extreme height appears to be 48 feet, but the average height of great waves is about 30 feet.

The heights of waves in different seas have been estimated as follows:—

In North Atlantic, from 24 to 43 feet; Bay of Biscay, 36 feet; Pacific Ocean, 32 feet; South Atlantic, 22 feet; Mediterranean, 15 feet; German Ocean, 13 feet; around the British coast, about 8 or 9 feet.

ARTICLE 485, p. 728.

DRAINAGE.—Centrifugal pumps are much used for lifting water. About 400 feet per minute is allowed for the speed of the water through the pipes; the speed of the periphery of the wheel is five or six times this, and the diameter of the wheel is generally from two to three times the diameter of the pipes.

The small sized windmill, with fan-shaped circular sail, is useful for drainage or water lifting purposes. The old form of windmill (see *Manual of the Steam Engine and other Prime Movers*) with a fair wind gives about one-horse power with a sail of 31 feet diameter. The efficiency of the small circular wheels seems to be rather higher. In similarly formed mills the power varies approximately as the square of the diameter of the wheel.

Where sea water is obtainable it is found to be very suitable for flushing the main sewers of towns and laying the dust on the streets

ARTICLE 450, p. 685.

FLOW OF WATER.—The formula $v = 8\cdot025\sqrt{\frac{hd}{4fl}}$ given on p. 685, is only suitable for long lengths of pipes, where the ratio of l to h is great.

For very short lengths, where the ratio of l to h approaches to equality, the formula will take the form of $v = 8\cdot025\sqrt{\frac{hd}{d + 4fl}}$.

ARTICLE 7, p. 5.

ORDNANCE DATUM.—The datum, from which the levels noted on the ordnance plans, referred to at p. 5, are reckoned, is the mean tide-level at Liverpool, and may be stated as a point situated at a height of 4·67 feet above the level of the old dock sill at Liverpool. For Ireland it is a point on Poolbeg Lighthouse in Dublin Bay, representing the low-water mark of spring tides.

ARTICLE 386, p. 585.

PHOSPHOR BRONZE.—This metal, an alloy of copper, tin, and phosphorus, is found to possess a high tensile strength and great endurance as regards wear, and is being used principally for bearings and machinery.

The various strengths of this metal appear to vary greatly according to the proportions of materials used, hence different alloys are employed for different classes of work; the range of ultimate tensile strength being from 22,000 lbs. to 57,000 lbs. per square inch, whilst the ultimate compressive strength varies from about 21,500 lbs. to 45,000 lbs. per square inch.

It also appears that wire made of this metal possesses the quality of high tensile strength.

ARTICLE 495, p. 734.

THE ACCUMULATOR.—The application of hydraulic power for the transmission of pressure is of much importance to the engineer. Machines for rivetting, punching, and shearing iron plates can be worked by this means, and heavy loads, such as swing bridges, cranes, hoists, &c., are also readily moved by hydraulic machinery.

The pressures used are very high, and, to obtain these, recourse is had to an "accumulator." The accumulator is on the principle of the hydraulic press—viz., a movable weighted plunger, raised by the action of force-pumps. The movable part can be loaded with what weights are necessary, and raised by the pressure of the water forced into the cylinder in which the plunger works: the pressure of the water will then correspond to the load which it supports. By means of a separate pipe leading to another plunger or piston connected with, say, the die of a rivetting machine, hydraulic connection is maintained with the cylinder, and thus, when the stroke of the die is made, a corresponding fall of the weighted part also takes place, whereby a sudden expenditure of the work previously accumulated is brought about. Variations in pressure and lengths of stroke are necessary for different operations; in rivetting machines, however, a load of 5 tons, having a fall of about 2 feet, under a pump pressure of 1,500 lbs. per square inch, is sufficient with a travel of die of 2¼ inches to close up a rivet, the final pressure on the die being about 40 tons.

HYDRAULIC RAM.—By means of the hydraulic ram, water at a low head may be made available for raising a portion of the same water to a higher level than the supply.

The action of this machine depends upon utilising part of the energy of the supply-water in closing a heavy clack-valve, and in compressing air in air-vessels connected by clack-valves. During this process a loss of water takes place, as the closing of the heavy clack-valve is dependent upon the increased velocity of flow due to this waste. When the valve shuts, the outflow is stopped, and the energy of the water is now spent in compressing the air in the air-vessels, by which means part of the water is forced upwards through the delivery-pipe. As the velocity of the water has now been reduced, the heavy clack-valve falls, and the process is repeated.

As the efficiency of such a machine will be the useful work performed in raising the water above the reservoir, divided by the work due to the fall of the water run to waste, the ratio $\frac{q\,h}{Q\,H}$ will represent the efficiency, where H = fall from reservoir to ram, h = height of delivery above reservoir, Q = quantity run to waste, q = quantity delivered. The results as to efficiency differ considerably, depending on mechanism, and proportion of height of fall and height of delivery; the best results appearing to be obtained when the proportion is as 1 : 10; the efficiency so obtained will be about ·6, and consequently $q = \frac{6\,Q\,H}{h}$.

Article 354, p. 503.

The floating of solid on molten cast iron, referred to at p. 503, arises from increase in bulk of the solid due to rapid expansion on being placed in the molten iron. This can be readily proved by measuring the solid piece before and after it has been in contact with the molten iron. This expansion is at least equal to the whole contraction in cooling from the molten to the cold solid condition.

No conclusion, however, can be directly drawn from this in favour of expansion in the mould on setting, as the temperature of the floating solid pieces, if removed immediately from the molten iron, is much below that of the melting point, as shown by pieces of lead remaining unmelted when placed upon them, and also by these retaining their crystalline character when broken across. The sharpness of an iron casting is due principally to the fluidity of the metal and condition of the mould. The breaking of the skin of a piece of cast iron under stress does not seem necessarily to weaken the piece, as in many cases test-bars, with the skin wholly removed by planing, give quite as good results as those cast from the same iron, but left untouched.

Article 225, p. 370.

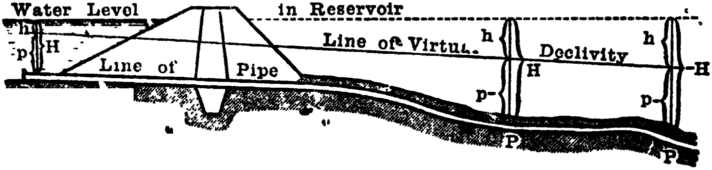
In testing cements by making up balls from a stiff paste of the material, it is necessary to let the ball stand in air for a short time so as to obtain a skin before placing it in the water. Thus, with a hot or quick setting Portland cement, the ball may require to stand for about ten minutes, whilst with a cool or slow setting cement, at least double this time is required.

Cements are tested for strength by mixing a quantity of the material in a mould with water, so as to form a stiffish paste; after a short exposure in air it is placed in water for seven days, at the expiration of which, the now hardened cement or test-brick can be broken by tension.

Good Portland cement, when treated in this way, should give at least 350 lbs. per square inch as the tensile strength.

Articles 446, p. 674; 447, p. 678; and 477, p. 722.

The *hydraulic* pressure may be briefly stated as the difference between the hydrostatic pressure and the loss of head. These values being taken, say in feet, at any point in the line of pipe.



If these values of hydraulic pressure be set off vertically from such points in the line of pipe, and a line drawn joining their extremities, this will be the *line of virtual declivity*.

In the annexed fig., H is the hydrostatic pressure, h the loss of head, and p the hydraulic pressure, and therefore $p = H - h$. P being any point in line of pipe.

Example.—Let $h = \frac{v^2 l}{2500 d}$. (See (2), p. 685.)

Then we have $p = H - \frac{v^2 l}{2500 d}$. Let $H = 200$ ft., $l = 6000$ ft., $d = \frac{1}{4}$ foot, and $v = 2$ ft. per second.

Then $p = 200 - \frac{4 \times 6000 \times 4}{2500} = 161{\cdot}6$ ft. as the hydraulic pressure.

Article 477, p. 723.

Water-pipes made of wrought-iron plates rivetted together have been tried in America, notably at San Francisco, where a main line of pipes, varying from 30 to 44 inches in diameter, was laid for a distance of 28 miles. The pipes were made in 28 feet lengths.

These were coated by means of a natural pitch got in the district.

Thin steel plates have also been used for forming water-pipes. The durability of such pipes of thin metal, doubtless, depends upon the completeness of the coating with which their surfaces are protected.

Article 406, p. 607.

Frictional Resistance.—During the sinking of iron cylinders to form the piers of bridges, it is found that the weights required to overcome the frictional resistance of the material through which the cylinders are forced, is about 3 cwt. per square foot in stiff clay, and about 2 cwt. per square foot in opener material, such as sand and stones with mud.

Articles 238 and 525, pp. 380 and 765.

Foundations.—Pressure at bottoms of cylinders carrying quay walls when sunk in sand $2\frac{3}{4}$ tons per square foot, and where the cylinders carry swing bridges or cranes, from $3\frac{1}{2}$ tons to $4\frac{1}{2}$ tons per square foot. In ordinary quay walls, without cylinders, founded upon stiff clay, about 2 tons per square foot.

Article 360, p. 515.

Rivets.—From some recent experiments on the strength of steel rivets it appears that with a tensile strength of about 25 tons in the rivet, the shearing strength was about 22·7 tons. Low tensile strength favours safety as the rivet is less likely to suffer from shocks

Article 438, p. 664.

Permanent Way.—In railways of narrow gauge, such as 3 feet 6 inches, the rails may weigh 35 lbs. per yard, when spiked to the sleepers, as is commonly done abroad. The latter are spaced about 2 feet 8 inches. If wood is used for sleepers rapid decay frequently sets in unless protected by some chemical process (see p. 453, on the Preservation of Timber). When used, unprotected abroad, pitch pine, where readily obtainable, has proved very serviceable. The sleepers may be about 7 feet long by 8 inches broad and 5 inches deep.

Article 498, p. 738.

Water Supply.—Where filtration is not considered necessary it is usual to provide copper wire strainers to keep back impurities from entering the pipes. The fineness of the mesh will depend on the character of the water. From practice the quantity passed through straining cloth of the finer meshes is about 16 gallons per minute per square foot. Parallel wire straining cloth is also used for this purpose, it being more readily cleaned than square meshed cloth. From some experiments with cloth of 39 square inches per superficial foot, it appears that square mesh gives 62 gallons per minute with $\frac{1}{4}$ inch head, and parallel mesh gives 110 gallons under the same conditions.*

* See *Trans. Inst. Engineers and Shipbuilders in Scotland*, vol. xxxiii.

Article 162, p. 250.

Fracture of Cast-Iron.—The form of fracture shown in cast-iron test bars is usually either *straight* or *curved*. When a test bar breaks at the middle of the span by a load acting at that point, the line of fracture is *straight* and at right angles to the line of span; when the fracture occurs at any point removed from the middle of the span, the line of fracture is *curved*, and invariably points towards the point of application of load. The fracture doubtless commences at the extended side of the bar, and runs towards the side under compression; its course being determined by the spring of the unequal parts of the bar as they separate during fracture.

Article 477, p. 721.

Cast-Iron Water Pipes.—The following table shows the dimensions and weights of some of the principal sizes of cast-iron pipes including socket:—

Diameter.	Thickness.	Length		Weight.		
Inches.	Inches	Feet.	Inch.	Cwts	Qrs.	Lbs.
3	$\frac{3}{8}$	9	3	1	0	14
4	$\frac{3}{8}$	9	3	1	1	23
6	$\frac{7}{16}$	9	$3\frac{1}{2}$	2	2	0
8	$\frac{1}{2}$	9	4	3	2	26
10	$\frac{9}{16}$	9	4	5	1	14
12	$\frac{5}{8}$	9	$4\frac{1}{4}$	6	3	23
15	$\frac{11}{16}$	9	$4\frac{1}{2}$	9	1	26
18	$\frac{11}{16}$	12	$4\frac{1}{2}$	14	3	6
24	$\frac{3}{4}$	12	5	21	2	3
30	1	12	5	36	0	25
36	1	12	5	41	3	23

Article 341, p. 475.

Timber Bridges.—The life of such bridges for railway purposes is very varied, depending upon the situation, the nature of the wood employed, and its treatment. Thus some of the carefully made old timber bridges in Britain have only been replaced by iron after a life of nearly 40 years. Generally speaking, 25 or 30 years would be a likely average for the older bridges, but at the present time from 14 to 20 years, at home, may be nearer the limit. In the Colonies, large renewals are expected to be required in about 15 years after erecting.

Creosoting the timber has an important influence in extending the life of the structure.

The laminated arch and truss system are both adopted; the latter system offers facilities for adjustment required by change of form.

The following are the dimensions of some of the older railway timber arches of 60 feet span and a rise at centre of 9 feet 9 inches:—There were four ribs, the lower member being 2 feet 9 inches × 1 foot 3 inches, composed of eleven planks, each 15 × 3, bolted together and bent to the curve, the ends resting in cast-iron sockets in the piers. The upper member was a horizontal beam 13 × 9, resting on the crown of the arch and continued to, and resting on, the masonry piers. These members were connected by struts and ties.

On the top of these ribs there were cross beams, 10 × 6 and spaced 4 feet, projecting over the outer ribs and giving a width between handrails of 26 feet. Above these cross beams longitudinal runners, 14 × 6, carried the rails.*

* See *Trans. Inst. Engineers and Shipbuilders in Scotland*, vol. xxxiv.

Article 501, p. 742.

The Manchester Ship Canal.—This canal is $35\frac{1}{2}$ miles long.

The average width at water level is 172 feet, and at bottom 120 feet.

The depth is 26 feet.

There are numerous railway and road swing-bridges across the canal, strongly built of steel plates, having a tensile strength of from 27 to 31 tons per square inch, with an elongation of 20 per cent. in 8 inches. The rivet steel had a limit of 26 to 30 tons per square inch, with same elongation.

The maximum test load stress allowed in booms and tension bars was $6\frac{2}{3}$ tons per square inch and 5 tons on cross girders.

120 lbs. per square foot was allowed for swing-bridges, or a moving load of 40 tons on 4 wheels at 8 feet centres.

The cast steel used had an ultimate tensile strength of 26 to 32 tons per square inch, with an elongation of 10 per cent. on 8 inches. Bars, 1 inch square, bent double without fracture with a radius of $1\frac{5}{8}$ inch.

Article 496, p. 736.

Artesian Wells.*—"1. There must exist between the 'outcrop' and the well, at least, one continuous stratum sufficiently porous to permit the easy flow of water through it, and this stratum must be sandwiched between two impermeable beds thick enough to confine the water. The impermeable beds must be quite free from any defect that would permit the escape of water at some level lower than the mouth of the well in such quantity as would be inimical to a steady flow therefrom. The perfect condition of the upper confining beds is of much more importance than an unblemished condition of the lower confining beds, as it is likely the water in its downward course would meet with some stratum that would arrest its further progress.

"2. That the outcrop of the porous stratum exposed to saturation must be porous enough to readily admit the entrance of water, and its area must be ample for keeping the porous stratum always charged with water to the degree necessary for maintaining a copious flow equal, at least, to all reasonable demands made on the well.

"3. That the altitude of the outcrop must be high enough above the mouth of the well, so that the pressure due to the difference of level between them will, after providing for frictional resistance to the flow of the water in the porous stratum, be sufficient to cause the water to rise in the well and flow at the surface. Frictional resistance is by no means inconsiderable if a long distance intervenes between the outcrop and the well; its coefficient depends chiefly upon the degree of porosity of the water-bearing beds; hence its measurement is impracticable, and consequently it is always an unknown quantity.

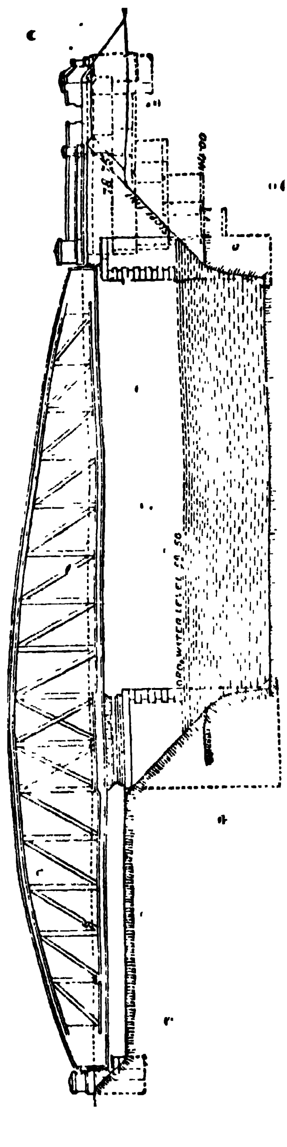
"4. That the rainfall in the region of the outcrop must be ample, and that the portion which enters the porous bed will be quite sufficient to ensure a steady and abundant supply of water to the well."

Article 227, p. 372.

Portland cement is specified by engineers to stand various tests, such as for fineness, strength, and weight.

The test for fineness is made by passing the cement through a sieve containing a certain number of meshes per square inch. Sieves containing 2,500 and 5,776 meshes per square inch are most commonly used, and from experiments it appears that, in passing various qualities of Portland cement through them, the average residue was about 8 per cent. with the 2,500 mesh and about 15 per cent. with the 5,776 mesh.

* Report of the Hydraulic Engineer on Water Supply, Queensland.



Where the cement was fine ground, these percentages became about $3\frac{1}{2}$ and $10\frac{1}{3}$.

The test for strength is by mixing the cement with water in a mould and allowing the briquettes so formed to lie in water for seven days, after which they should stand without breaking a tensile stress of 350 lbs. per square inch.

About 112 lbs. per bushel may be taken as the weight test.

Portland cement when new is hot and quick setting, and should be exposed in bulk to cool and air slake.

In using cement mortar a most important item is the quality of the sand, which should be clean, sharp, and rather coarse.

Tests are sometimes specified to show a tensile strength of 160 lbs. per square inch; the briquettes being made up of 1 part of cement and 3 parts of sand, and immersed for twenty-eight days in water.

Earthy, muddy, or shaley particles in the sand are objectionable.

Article 226, p. 371.

Some hydraulic limes afford good natural cements, and appear to increase in strength with lapse of time, attaining equality with Portland cement. As in the case of Portland cement, fine grinding is a necessity.

Article 505, p. 745.

Swing-Bridges.*—The fig. shows one of the swing-bridges on the Manchester Ship Canal, and is generally of the following dimensions:—Spans, 139 feet and 96 feet 10 inches; depth, 28 feet, decreasing to 7 feet 9 inches and 9 feet; width, 22 feet 8 inches.

The main girders of the swing-bridges are carried on cross girders resting on annular girders, and cast-iron conical rollers transmit the weight to the lower roller path.

The movement is by hydraulic power, the pressure used being 700 lbs. per square inch.

The flooring is made up of buckle-plates, levelled up with concrete, covered with an inch of sand, and $\frac{3}{4}$ inch of asphalte, 5-inch wood setts forming the roadway.

* *Trans. Inst. Engineers and Shipbuilders in Scotland*, vol. xxxvi.

ARTICLE 416, p. 624.

HORSE HAULAGE.—On a level surface a horse will probably draw on a tramway about five times the load that can be drawn on a good road. When the surfaces are inclined this advantage diminishes.

ARTICLE 427, p. 632.

ELECTRIC TRACTION.—The efficiency of the electric motor is about 80 per cent. of the energy supplied. The method of transmission of the electric energy may be by overhead wires on the trolley system, the rails acting as return conductors, or it may be by the conduit or underground system, where there are double wires.

ARTICLE 430, p. 641.

LIGHT RAILWAYS, so-called, may have any gauge up to the standard gauge of 4 feet 8½ inches. In some European countries the governments have adopted a gauge of 60 centimetres, or about 1 foot 11½ inches, for light railways for military purposes. (See also p. 792.)

ARTICLE 477, p. 722.

CORROSION IN WATER PIPES.—Pipes for the conveyance of water suffer much from corrosion. In the earlier days of water supply the cast-iron pipes used were at first laid without any protective coating, and, therefore, incrustation became unavoidable. Even now, with the excellent bituminous coating (see p. 795) which is always applied to the pipes, incrustation sets in, and after some years, depending upon the character of the water, the deposit attains such a thickness as to seriously diminish the delivery of the water, hence either a new main has to be laid, or recourse must be had to scraping, so as to get rid of the obstructions.

The form frequently taken by the deposit is that of roughly rounded patches; these seem to originate through some small defect in the coating, possibly at parts of the surface where rust may have been induced before coating. It is, therefore, of great importance during the manufacture of water pipes to see that they are free from rust before dipping into the hot solution of preservative composition.

To insure this, the pipes are either painted over with oil immediately after being cleaned, and freed from the core and mould, or they are proved with oil, and not with water, as in the usual method.

The scraping of water pipes dates from 1866, when an apparatus was tried at Torquay, the mains for the supply of this town having become much incrusted. The result of this experiment and some others showed the advantages to be obtained from such a process, and now the scraping of mains is arranged for in many cases by the introduction at certain parts of the line of piping of special castings, called hatch-boxes, to enable the scraper to be introduced and removed from the pipes.

The scraper consists essentially of a jointed arrangement of pistons and scraping faces or knives, the latter being pressed by springs against the inner surface of the pipe.

The pressure of the flowing water on the piston or stiffened leather discs on the after part of the machine drives the whole forward, and the knives act upon any incrustation met with. The scrapers are fitted so as to yield to any fixed obstruction, such as lead which may have run in from a lead-made joint.

ARTICLE 502, p. 742.

CANALS.—The speed of haulage by horses may be about 2 miles per hour, but in some cases wire ropes or chains are used; these lie at the bottom of the canal, and are gripped by a wheel at the bow. The average speed with this arrangement of a train of boats may be about 2¼ miles per hour.

ARTICLE 393, p. 596.

TUNNELS.—The Simplon Tunnel measures 12 miles long, and is 7000 feet below summit of mountain. Gradient from each end, 1 in 100. There are two tunnels 50 feet apart; the width is 16 feet, and height 7 feet. Hydraulic pressure was used for the excavating drills, and compressed air for ventilation.

ARTICLE 408, p. 611.

CAISSONS.—Caissons for docks may be floating or sliding. The floating caissons are divided and the lower part ballasted. Air and water chambers are provided for regulating the movements into position.

Sliding caissons require recesses in the dock walls for withdrawal when the dock is open.

ARTICLE 487, p. 729.

SEWAGE PURIFICATION.—In the purification of the sewage of large towns, successful attempts have been made. The discharges from the main sewers are led into chambers where screens remove floating matter. Catch pits then intercept the solid particles. The resulting fluid is then raised by pumps to mixing pits, lime is added, and the sludge is pressed into cakes. By filtration through coke and sand filters, the fluid is finally rendered quite pure.

SECTION IV., p. 688.

ELECTRIC TRACTION. Electricity is now applied in many cases where locomotives were used, especially on suburban lines, in tunnels such as that below the Mersey, and in the "Tubes" of London and Underground of Paris. In New York, the Elevated Railway has also become electrified. Besides the advantages obtained from absence of smoke, the trains can be run quicker through there being less time lost from starting to the required full rate of travel.

ARTICLE 442A, p. 671.

Snow and sand drift cause, on some railway lines, much obstruction, and various methods have been devised to meet the difficulty. The erection of fences or the planting of hedges on the windward side of cuttings where accumulations of drift readily take place serves for a time only, as the fence gets gradually covered up by the snow or sand, which then blows over and onward as before. An ingenious system of "blower" has been introduced on some lines subjected to drift obstructions, whether of sand or snow. The blower consists of two rows of timber posts placed along the windward slope of the cutting, wooden planking is fixed on the tops of the posts so as to form a roof sloping downwards, the angle being such that the direction of slope, if continued, would strike the line of rails in the cutting. The wind carrying the snow or sand strikes the underside of the roof of the blower, by which it is deflected downwards, and issues with sufficient force to carry the drift right across the rails to the leeward side of the cutting.

ARTICLE 518, p. 760.

GROINS.—It has been found that low groins are more suitable for giving stability to a beach than high groins, as the latter tend to cause scour due to the greater obstruction of the currents.

Low groins, having rows of posts placed at right angles to the line of coast and extending downwards to low water, with connecting planking a foot or two in depth seem to be effective in causing deposit of lateral drift.

REINFORCED CONCRETE, OR FERRO-CONCRETE.—This is a combination of concrete and steel or iron framing, the latter embedded in the concrete, and has been employed for various purposes, such as house walls, beams, arches, pillars, &c. One of the advantages is lightness of construction. Great care has to be taken that both the material, proportions, and mixing of the concrete shall be carefully attended to.

Town Refuse Destructors.—In many large towns the refuse gathered from the houses and streets is conveyed to depôts where it is consumed in furnaces, and the heat developed is utilised in the raising of steam whereby electric energy may be developed and transmitted to distant points for the performance of useful work. The refuse may be generally divided into water, combustible matter, and non-combustible in about equal proportions. The incombustible is mainly clinker, which may be afterwards used for the making of roads and for building material. It can be ground or crushed for mortar or for the making of tar pavements. Dust catchers are placed between the boiler and the chimney, and it, therefore, does not seem necessary to erect very high chimneys for the removal of the gaseous products of combustion.

Bridges.—The Zambezi Bridge, formally opened during the meeting of the British Association in South Africa, 1905, spans the Zambezi River below the Victoria Falls, and carries the Cape to Cairo Railway line across the deep gorge through which the river flows at that point, the height of the arch above the water being about 400 feet.

The form is that of an arch built out from bank to bank on the cantilever system. The span is 650 feet.

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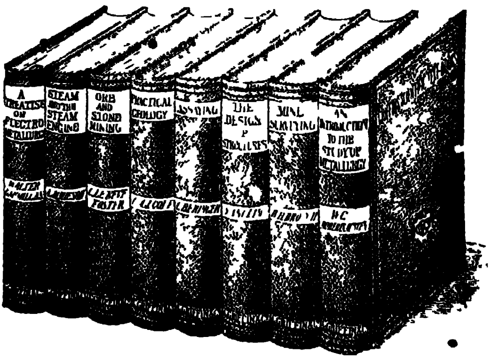

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